PROTECO LANDFILL
REACH "I" DRAINAGE CALCULATIONS

SLOPE = 11.8%

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
1.29	0.035	0.118	0.1	0.44	4.82	2.94
4.28	0.035	0.118	0.2	0.96	5.65	4.46
8.79	0.035	0.118	0.3	1.56	6.47	5.64
14.85	0.035	0.118	0.4	2.24	7.30	6.63
22.51	0.035	0.118	0.5	3.00	8.12	7.50
31.86	0.035	0.118	0.6	3.84	8.95	8.30
42.99	0.035	0.118	0.7	4.76	9.77	9.03
50.56	0.035	0.118	0.76	5.35	10.27	9.45
51.89	0.035	0.118	0.77	5.45	10.35	9.52
53.23	0.035	0.118	0.78	5.55	10.43	9.59
54.60	0.035	0.118	0.79	5.66	10.51	9.65
55.99	0.035	0.118	0.8	5.76	10.60	9.72
70.95	0.035	0.118	0.9	6.84	11.42	10.37
87.95	0.035	0.118	1	8.00	12.25	10.99
107.10	0.035	0.118	1.1	9.24	13.07	11.59
128.49	0.035	0.118	1.2	10.56	13.90	12.17
152.18	0.035	0.118	1.3	11.96	14.72	12.72

PROTECO LANDFILL
REACH "I" DRAINAGE CALCULATIONS
SLOPE = 10.5%

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
1.21	0.035	0.105	0.1	0.43	4.63	2.81
3.94	0.035	0.105	0.2	0.92	5.26	4.29
7.99	0.035	0.105	0.3	1.47	5.90	5.44
13.33	0.035	0.105	0.4	2.08	6.53	6.41
19.98	0.035	0.105	0.5	2.75	7.16	7.26
27.97	0.035	0.105	0.6	3.48	7.79	8.04
37.35	0.035	0.105	0.7	4.27	8.43	8.75
48.19	0.035	0.105	0.8	5.12	9.06	9.41
54.16	0.035	0.105	0.85	5.57	9.38	9.73
55.41	0.035	0.105	0.86	5.66	9.44	9.79
56.66	0.035	0.105	0.87	5.75	9.50	9.85
60.53	0.035	0.105	0.9	6.03	9.69	10.04
74.43	0.035	0.105	1	7.00	10.32	10.63
89.95	0.035	0.105	1.1	8.03	10.96	11.20
107.15	0.035	0.105	1.2	9.12	11.59	11.75
126.08	0.035	0.105	1.3	10.27	12.22	12.28

PROTECO LANDFILL
REACH "I" DRAINAGE CALCULATIONS
SLOPE = 10.5%, n=0.038

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
1.11	0.038	0.105	0.1	0.43	4.63	2.58
3.63	0.038	0.105	0.2	0.92	5.26	3.95
7.36	0.038	0.105	0.3	1.47	5.90	5.01
12.28	0.038	0.105	0.4	2.08	6.53	5.90
18.40	0.038	0.105	0.5	2.75	7.16	6.69
25.76	0.038	0.105	0.6	3.48	7.79	7.40
34.40	0.038	0.105	0.7	4.27	8.43	8.06
44.38	0.038	0.105	0.8	5.12	9.06	8.67
52.19	0.038	0.105	0.87	5.75	9.50	9.08
53.36	0.038	0.105	0.88	5.84	9.57	9.13
54.55	0.038	0.105	0.89	5.94	9.63	9.19
55.75	0.038	0.105	0.9	6.03	9.69	9.25
68.55	0.038	0.105	1	7.00	10.32	9.79
82.85	0.038	0.105	1.1	8.03	10.96	10.32
98.69	0.038	0.105	1.2	9.12	11.59	10.82
116.13	0.038	0.105	1.3	10.27	12.22	11.31

PROTECO LANDFILL
REACH "I" DRAINAGE CALCULATIONS
SLOPE = 11.8%, n=0.038

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
1.19	0.038	0.118	0.1	0.44	4.82	2.71
3.94	0.038	0.118	0.2	0.96	5.65	4.11
8.10	0.038	0.118	0.3	1.56	6.47	5.19
13.67	0.038	0.118	0.4	2.24	7.30	6.10
20.73	0.038	0.118	0.5	3.00	8.12	6.91
29.34	0.038	0.118	0.6	3.84	8.95	7.64
39.60	0.038	0.118	0.7	4.76	9.77	8.32
51.57	0.038	0.118	0.8	5.76	10.60	8.95
52.86	0.038	0.118	0.81	5.86	10.68	9.01
54.18	0.038	0.118	0.82	5.97	10.76	9.08
55.51	0.038	0.118	0.83	6.08	10.84	9.14
56.86	0.038	0.118	0.84	6.18	10.93	9.20
65.34	0.038	0.118	0.9	6.84	11.42	9.55
81.01	0.038	0.118	1	8.00	12.25	10.13
98.65	0.038	0.118	1.1	9.24	13.07	10.68
118.34	0.038	0.118	1.2	10.56	13.90	11.21
140.17	0.038	0.118	1.3	11.96	14.72	11.72

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, P.R.	Drainage Swales

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JAL	9/12/94	MLP	9/21/94		

$$Q = 6.03\left(\frac{1.49}{0.038}\right)\left(\frac{6.03}{9.69}\right)^{2/3}(0.105)^{1/2}$$

$$Q = 6.03\,(39.21)\,(0.73)\,(0.324)$$

$$Q = 55.76 \text{ cfs} \quad \text{ok}$$

Check Reach "I" spreadsheet for $S = 11.8\%$, $n = 0.038$, $d = 0.82'$

$$A = 4(d) + d\,(4d)$$

$$A = 4(0.82) + 0.82\,(4(0.82))$$

$$A = 3.28 + 2.69 = 5.97 \text{ ft}^2$$

$$P = 4 + (2)\sqrt{(0.82)^2 + (4(0.82))^2}$$

$$P = 4 + 6.76 = 10.76$$

$$Q = 5.97\left(\frac{1.49}{0.038}\right)\left(\frac{5.97}{10.76}\right)^{2/3}(0.118)^{1/2}$$

$$Q = 5.97\,(39.21)\,(0.67)\,(0.34)$$

$Q = 54.19$ cfs <u>ok</u>

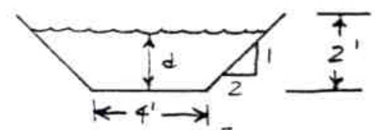
OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO		Location Penuelas, P.R.		Subject Drainage Swales	
Preparer's Initials JAL	Date 9/13/94	Reviewer's Initials MLP	Date 9/21/94	Approver's Initials		Date

<u>Reach "J" Drainage Swale</u>

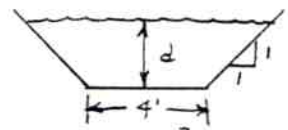


Min $S = \frac{2}{8} = 25\%$

$Q = Q_{Reach\ "H"} + Q_{Reach\ "I"}$

$Q = 23.6$ cfs $+ 55.7$ cfs

$Q = 79.3$ cfs



Max $S = \frac{2}{5} = 40\%$

Check Min. Slope section for capacity

$A = 4(d) + d(2d)$

$P = 4 + (2)\sqrt{d^2 + (2d)^2}$

From spreadsheet w/ $S = 25\%$

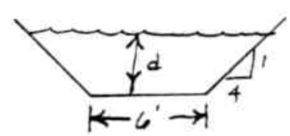
$d \cong 0.86$ ft, Total depth = 2' <u>ok</u>

Calculate velocity at max. slope for rip rap sizing

From spreadsheet w/ $S = 40\%$,

Velocity $\cong 18.7$ fps

Velocity too high, reconfigure channel



$A = 6(d) + d(4d)$

$P = 6 + (2)\sqrt{d^2 + (4d)^2}$

$S = 40\%$

Use $n = 0.041$, From spreadsheet,

Max velocity $\cong 14.4$ fps

$\therefore$ Use R-7 rip rap

COMPUTATION SHEET

OHM Corporation

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Penuelas, P.R.	Subject Drainage Swales

Preparer's Initials JAL	Date 9/13/94	Reviewer's Initials MLP	Date 9/21/94	Approver's Initials	Date

Check flow depth

From spreadsheet w/ Slope = 25%, n = 0.041

Flow depth ≅ 0.74 feet ok

Check spreadsheet for Slope = 25%, n = 0.041, d = 0.74 feet

$A = 6(d) + d(4d)$

$A = 6(0.74) + 0.74(4(0.74))$

$A = 4.44 + 2.19 = 6.63$ ok

$P = 6 + (2)\sqrt{d^2 + (4d)^2}$

$P = 6 + (2)\sqrt{0.74^2 + (4(0.74))^2}$

$P = 6 + 6.10 = 12.10$ ok

$Q = 6.63\left(\frac{1.49}{0.041}\right)\left(\frac{6.63}{12.10}\right)^{2/3}(0.25)^{1/2}$

$Q = 6.63(36.34)(0.67)(0.5)$

$Q = 80.51$ ok

Check spreadsheet for Slope = 40%, n = 0.041, d = 0.65 ft.

$A = 6d + d(4d)$

$A = 6(0.65) + 0.65(4(0.65))$

$A = 3.9 + 1.69 = 5.59$ ok

PROTECO LANDFILL
REACH "J" DRAINAGE CALCULATIONS
SLOPE = 25%, n=0.038

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
1.69	0.038	0.25	0.1	0.42	4.45	4.03
5.60	0.038	0.25	0.2	0.92	5.26	6.09
11.36	0.038	0.25	0.3	1.47	5.90	7.73
18.95	0.038	0.25	0.4	2.08	6.53	9.11
28.39	0.038	0.25	0.5	2.75	7.16	10.32
39.75	0.038	0.25	0.6	3.48	7.79	11.42
53.09	0.038	0.25	0.7	4.27	8.43	12.43
68.48	0.038	0.25	0.8	5.12	9.06	13.38
76.98	0.038	0.25	0.85	5.57	9.38	13.83
78.74	0.038	0.25	0.86	5.66	9.44	13.92
80.53	0.038	0.25	0.87	5.75	9.50	14.00
82.34	0.038	0.25	0.88	5.84	9.57	14.09
86.02	0.038	0.25	0.9	6.03	9.69	14.27
105.78	0.038	0.25	1	7.00	10.32	15.11
127.84	0.038	0.25	1.1	8.03	10.96	15.92
152.28	0.038	0.25	1.2	9.12	11.59	16.70
179.19	0.038	0.25	1.3	10.27	12.22	17.45

PROTECO LANDFILL
REACH "J" DRAINAGE CALCULATIONS
SLOPE = 40%, n=0.038

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
2.11	0.038	0.4	0.1	0.41	4.28	5.15
6.70	0.038	0.4	0.2	0.84	4.57	7.98
13.18	0.038	0.4	0.3	1.29	4.85	10.21
21.31	0.038	0.4	0.4	1.76	5.13	12.11
30.98	0.038	0.4	0.5	2.25	5.41	13.77
42.12	0.038	0.4	0.6	2.76	5.70	15.26
54.67	0.038	0.4	0.7	3.29	5.98	16.62
68.62	0.038	0.4	0.8	3.84	6.26	17.87
76.11	0.038	0.4	0.85	4.12	6.40	18.46
77.65	0.038	0.4	0.86	4.18	6.43	18.58
79.20	0.038	0.4	0.87	4.24	6.46	18.69
80.76	0.038	0.4	0.88	4.29	6.49	18.81
83.94	0.038	0.4	0.9	4.41	6.55	19.03
100.63	0.038	0.4	1	5.00	6.83	20.13
118.69	0.038	0.4	1.1	5.61	7.11	21.16
138.11	0.038	0.4	1.2	6.24	7.39	22.13
158.92	0.038	0.4	1.3	6.89	7.68	23.07

PROTECO LANDFILL
REACH "J" DRAINAGE CALCULATIONS
SLOPE = 25%, n=0.041

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
2.38	0.041	0.25	0.1	0.64	6.82	3.72
7.77	0.041	0.25	0.2	1.36	7.65	5.71
15.71	0.041	0.25	0.3	2.16	8.47	7.27
26.12	0.041	0.25	0.4	3.04	9.30	8.59
39.02	0.041	0.25	0.5	4.00	10.12	9.75
54.46	0.041	0.25	0.6	5.04	10.95	10.81
72.53	0.041	0.25	0.7	6.16	11.77	11.77
76.46	0.041	0.25	0.72	6.39	11.94	11.96
78.47	0.041	0.25	0.73	6.51	12.02	12.05
80.51	0.041	0.25	0.74	6.63	12.10	12.14
93.30	0.041	0.25	0.8	7.36	12.60	12.68
116.88	0.041	0.25	0.9	8.64	13.42	13.53
143.35	0.041	0.25	1	10.00	14.25	14.33
172.82	0.041	0.25	1.1	11.44	15.07	15.11
205.39	0.041	0.25	1.2	12.96	15.90	15.85
241.15	0.041	0.25	1.3	14.56	16.72	16.56

PROTECO LANDFILL
REACH "J" DRAINAGE CALCULATIONS
SLOPE = 40%, n=0.041

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
3.01	0.041	0.4	0.1	0.64	6.82	4.71
9.83	0.041	0.4	0.2	1.36	7.65	7.23
19.87	0.041	0.4	0.3	2.16	8.47	9.20
33.04	0.041	0.4	0.4	3.04	9.30	10.87
49.35	0.041	0.4	0.5	4.00	10.12	12.34
68.89	0.041	0.4	0.6	5.04	10.95	13.67
77.62	0.041	0.4	0.64	5.48	11.28	14.17
79.89	0.041	0.4	0.65	5.59	11.36	14.29
82.19	0.041	0.4	0.66	5.70	11.44	14.41
91.74	0.041	0.4	0.7	6.16	11.77	14.89
118.02	0.041	0.4	0.8	7.36	12.60	16.03
147.84	0.041	0.4	0.9	8.64	13.42	17.11
181.32	0.041	0.4	1	10.00	14.25	18.13
218.60	0.041	0.4	1.1	11.44	15.07	19.11
259.79	0.041	0.4	1.2	12.96	15.90	20.05
305.03	0.041	0.4	1.3	14.56	16.72	20.95

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, PR	Drainage Swales

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JAL	9/13/94	MLR	9/21/94		

Check $S = 40\%$, $n = 0.041$, $d = 0.65$ (con't)

$$P = 6 + (2)\sqrt{d^2 + (4d)^2}$$

$$P = 6 + (2)\sqrt{(0.65)^2 + (4(0.65))^2}$$

$$P = 6 + 5.36 = 11.36 \quad \underline{ok}$$

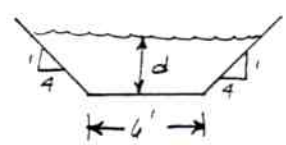
$$Q = 5.59\left(\frac{1.49}{0.041}\right)\left(\frac{5.59}{11.36}\right)^{2/3}(0.4)^{1/2}$$

$$Q = 5.59\,(36.34)\,(0.62)\,(0.63)$$

$$Q = 79.89 \quad \underline{ok}$$

<u>Reach "K" Drainage Swale</u>

$Q = 39$ cfs

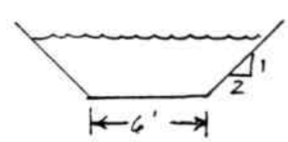


Check Min. Slope Section for Capacity

$A = 6d + d(4d)$

$P = 6 + (2)\sqrt{d^2 + (4d)^2}$

Min $S = \frac{2}{8} = 25\%$



From spreadsheet for Slope = 25%,

$d = 0.5'$; Actual channel depth $\cong 2.0$

$\therefore$ Capacity ok

Max $S = \frac{2}{6} = 33\%$

COMPUTATION SHEET

OHM Corporation

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, P.R.	Drainage Swales

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JAL	9/16/94	MLR	9/21/94		

Reach "K" cont'd

Calculate velocity at max slope for sizing rip rap

$A = 6d + d(2d)$

$P = 6 + \sqrt{d^2 + (2d)^2}$

From spreadsheet for $S = 33\%$, $n = 0.041$

Velocity $\approx$ 11.5 fps

$\therefore$ Use R-6 rip rap, $n = 0.0395$

With smaller n, depth of flow will be less, thus capacity calculation adequate. Verify adequacy of R-6 rip rap by calculating velocity at max. slope for $n = 0.0395$

Velocity $\approx$ 11.8 fps $\therefore$ R-6 rip rap ok

Check spreadsheet for slope = 25%, $n = 0.041$, $d = 0.5'$

$A = 6(d) + d(4d)$

$A = 6(0.5) + 0.5(4(0.5))$

$A = 3 + 1 = 4.0\ ft^2$

$P = 6 + (2)\sqrt{d^2 + (4d)^2}$

$P = 6 + (2)\sqrt{(0.5)^2 + (4(0.5))^2}$

$P = 10.12$

PROTECO LANDFILL
REACH "K" DRAINAGE CALCULATIONS
SLOPE = 33%, n=0.0395

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
2.80	0.0395	0.33	0.1	0.62	6.45	4.51
8.98	0.0395	0.33	0.2	1.28	6.89	7.01
17.83	0.0395	0.33	0.3	1.98	7.34	9.01
29.13	0.0395	0.33	0.4	2.72	7.79	10.71
38.42	0.0395	0.33	0.47	3.26	8.10	11.78
39.84	0.0395	0.33	0.48	3.34	8.15	11.93
42.75	0.0395	0.33	0.5	3.50	8.24	12.21
58.64	0.0395	0.33	0.6	4.32	8.68	13.57
76.78	0.0395	0.33	0.7	5.18	9.13	14.82
97.17	0.0395	0.33	0.8	6.08	9.58	15.98
119.81	0.0395	0.33	0.9	7.02	10.02	17.07
144.74	0.0395	0.33	1	8.00	10.47	18.09
171.97	0.0395	0.33	1.1	9.02	10.92	19.07
201.54	0.0395	0.33	1.2	10.08	11.37	19.99
233.48	0.0395	0.33	1.3	11.18	11.81	20.88

PROTECO LANDFILL
REACH "K" DRAINAGE CALCULATIONS
SLOPE = 33%, n=0.041

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
2.70	0.041	0.33	0.1	0.62	6.45	4.35
8.65	0.041	0.33	0.2	1.28	6.89	6.76
17.18	0.041	0.33	0.3	1.98	7.34	8.68
28.06	0.041	0.33	0.4	2.72	7.79	10.32
38.38	0.041	0.33	0.48	3.34	8.15	11.49
39.77	0.041	0.33	0.49	3.42	8.19	11.63
41.18	0.041	0.33	0.5	3.50	8.24	11.77
56.49	0.041	0.33	0.6	4.32	8.68	13.08
73.97	0.041	0.33	0.7	5.18	9.13	14.28
93.61	0.041	0.33	0.8	6.08	9.58	15.40
115.43	0.041	0.33	0.9	7.02	10.02	16.44
139.44	0.041	0.33	1	8.00	10.47	17.43
165.68	0.041	0.33	1.1	9.02	10.92	18.37
194.16	0.041	0.33	1.2	10.08	11.37	19.26
224.93	0.041	0.33	1.3	11.18	11.81	20.12

PROTECO LANDFILL
REACH "K" DRAINAGE CALCULATIONS
SLOPE = 25%, n=0.041

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
2.38	0.041	0.25	0.1	0.64	6.82	3.72
7.77	0.041	0.25	0.2	1.36	7.65	5.71
15.71	0.041	0.25	0.3	2.16	8.47	7.27
26.12	0.041	0.25	0.4	3.04	9.30	8.59
39.02	0.041	0.25	0.5	4.00	10.12	9.75
54.46	0.041	0.25	0.6	5.04	10.95	10.81
72.53	0.041	0.25	0.7	6.16	11.77	11.77
93.30	0.041	0.25	0.8	7.36	12.60	12.68
116.88	0.041	0.25	0.9	8.64	13.42	13.53
143.35	0.041	0.25	1	10.00	14.25	14.33
172.82	0.041	0.25	1.1	11.44	15.07	15.11
205.39	0.041	0.25	1.2	12.96	15.90	15.85
241.15	0.041	0.25	1.3	14.56	16.72	16.56

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Penuelas, P.R.	Subject Drainage Swales

Preparer's Initials JAL	Date 9/16/94	Reviewer's Initials MLP	Date 9/21/94	Approver's Initials	Date

$$Q = 4\left(\frac{1.49}{0.041}\right)\left(\frac{4}{10.12}\right)^{2/3}(0.25)^{1/2}$$

$$Q = 4(36.34)(.54)(.5)$$

$$Q = 39.02 \quad \underline{ok}$$

Check spreadsheet for $S = 33\%$, $n = 0.0395$, $d = 0.47$

$$A = 6d + d(2d)$$

$$A = 6(0.47) + 0.47(2(0.47))$$

$$A = 2.82 + 0.44$$

$$A = 3.26$$

$$P = 6 + (2)\sqrt{d^2 + (2d)^2}$$

$$P = 6 + (2)\sqrt{0.47^2 + (2(0.47))^2}$$

$$P = 8.1$$

$$Q = 3.26\left(\frac{1.49}{0.0395}\right)\left(\frac{3.26}{8.1}\right)^{2/3}(0.33)^{1/2}$$

$$Q = 3.26(37.72)(0.54)(0.57)$$

$$Q = 38.39 \text{ cfs} \quad \underline{ok}$$

OHM Corporation

COMPUTATION SHEET

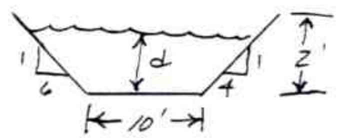
Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject	
16139	PROTECO	Penuelas, P.R.	Drainage Swales	
Preparer's Initials: JAL	Date: 9/16/94	Reviewer's Initials: MLP	Date: 9/21/94	Approver's Initials:	Date:

Existing Ditch "B"

Check capacity adjacent to Waste Unit 1

to verify run on prevention



$A = 10(d) + \frac{1}{2}(4d)(d) + \frac{1}{2}(6d)(d)$

$P = 10 + \sqrt{d^2+(4d)^2} + \sqrt{d^2+(6d)^2}$

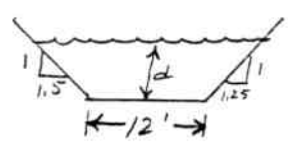
$S = \frac{4}{140} = 2.9\%$

From 2.9% slope spreadsheet, $d \cong 0.76'$, channel depth = 2'

$\therefore$ channel depth adequate

Calculate velocity at max slope for rip rap sizing.

Maximum velocity may occur at either maximum vertical slope or maximum side slopes. Check velocity in ditch between contours 380 & 378.



$A = 12(d) + \frac{1}{2}(1.25d)(d) + \frac{1}{2}(1.5d)(d)$

$P = 12 + \sqrt{d^2+(1.25d)^2} + \sqrt{d^2+(1.5d)^2}$

$S = \frac{2}{130} = 1.5\%$

From 1.5% slope spreadsheet, Velocity $\cong$ 4.9 fps

Velocity for 2.9% slope spreadsheet $\cong$ 5.3 fps

$\therefore$ Use R-3 rip rap, $n = 0.031$

PROTECO LANDFILL
EXISTING DITCH "B" DRAINAGE CALCULATIONS
SLOPE = 1.5%, n=0.031

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
0.76	0.031	0.015	0.1	0.61	6.32	1.23
2.41	0.031	0.015	0.2	1.26	6.66	1.92
4.76	0.031	0.015	0.3	1.92	7.02	2.47
7.69	0.031	0.015	0.4	2.62	7.40	2.94
11.16	0.031	0.015	0.5	3.34	7.80	3.34
15.11	0.031	0.015	0.6	4.10	8.22	3.69
19.52	0.031	0.015	0.7	4.87	8.66	4.00
24.35	0.031	0.015	0.8	5.68	9.12	4.29
29.57	0.031	0.015	0.9	6.51	9.60	4.54
35.17	0.031	0.015	1	7.38	10.10	4.77
38.70	0.031	0.015	1.06	7.90	10.41	4.90
39.30	0.031	0.015	1.07	7.99	10.46	4.92
39.90	0.031	0.015	1.08	8.08	10.52	4.94
41.12	0.031	0.015	1.1	8.26	10.62	4.98
47.41	0.031	0.015	1.2	9.18	11.16	5.16
54.02	0.031	0.015	1.3	10.12	11.72	5.34

PROTECO LANDFILL
EXISTING DITCH "B" DRAINAGE CALCULATIONS
SLOPE = 2.9%, n=0.031

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
1.08	0.031	0.029	0.1	0.65	7.02	1.66
3.55	0.031	0.029	0.2	1.40	8.06	2.53
7.21	0.031	0.029	0.3	2.25	9.13	3.20
12.04	0.031	0.029	0.4	3.20	10.21	3.76
18.05	0.031	0.029	0.5	4.25	11.31	4.25
25.28	0.031	0.029	0.6	5.40	12.43	4.68
33.74	0.031	0.029	0.7	6.65	13.58	5.07
38.45	0.031	0.029	0.75	7.31	14.15	5.26
39.43	0.031	0.029	0.76	7.45	14.27	5.29
40.42	0.031	0.029	0.77	7.58	14.39	5.33
43.48	0.031	0.029	0.8	8.00	14.74	5.44
54.54	0.031	0.029	0.9	9.45	15.92	5.77
66.93	0.031	0.029	1	11.00	17.12	6.08
80.71	0.031	0.029	1.1	12.65	18.35	6.38
95.91	0.031	0.029	1.2	14.40	19.59	6.66
112.55	0.031	0.029	1.3	16.25	20.85	6.93

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

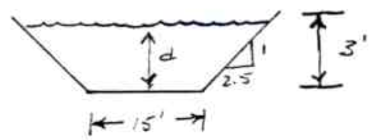
Proj. No. 16139	Client PROTECO		Location Penuelas, P.R.		Subject Drainage Swales	
Preparer's Initials JAL	Date 9/19/94	Reviewer's Initials MLP		Date 9/21/94	Approver's Initials	Date

<u>Reach "L" Drainage Swale</u>

$Q = Q_{Reach\ J} + Q_{Reach\ K} + Q_{Overland\ Flow\ \#2} + Q_{Detention\ Basin}$

$Q = 79.3\ cfs + 39\ cfs + 151.2\ cfs + 41.5\ cfs$

$Q = 311\ cfs$



Check min slope section for capacity

Min. $S = \frac{5}{105} = 4.8\%$

Assume $n = 0.038$

Max. $S = \frac{5}{70} = 7.1\%$

From spreadsheet w/ $S = 4.8\%$

Flow depth = 1.61 ft

Check max. slope for velocity to size rip rap

From spreadsheet w/ $S = 7.1\%$, velocity = 11.6 fps

∴ R-6 rip rap required; $n = 0.0395$ fps

Check flow depth w/ $S = 4.8\%$, $n = 0.0395$

from spreadsheet, flow depth = 1.64 ft ∴ depth adequate

Check velocity w/ $n = 0.0395$, $S = 7.1\%$

from spreadsheet, velocity = 11.3 fps

PROTECO LANDFILL
REACH "L" DRAINAGE CALCULATIONS
SLOPE = 4.8%, n=0.038

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter (ft)	Velocity (fps)
2.77	0.038	0.048	0.1	1.53	15.54	1.81
8.84	0.038	0.048	0.2	3.10	16.08	2.85
17.48	0.038	0.048	0.3	4.73	16.62	3.70
28.40	0.038	0.048	0.4	6.40	17.15	4.44
41.44	0.038	0.048	0.5	8.13	17.69	5.10
56.49	0.038	0.048	0.6	9.90	18.23	5.71
73.49	0.038	0.048	0.7	11.73	18.77	6.27
92.38	0.038	0.048	0.8	13.60	19.31	6.79
113.13	0.038	0.048	0.9	15.53	19.85	7.29
135.72	0.038	0.048	1	17.50	20.39	7.76
160.13	0.038	0.048	1.1	19.53	20.92	8.20
186.35	0.038	0.048	1.2	21.60	21.46	8.63
214.38	0.038	0.048	1.3	23.73	22.00	9.04
244.21	0.038	0.048	1.4	25.90	22.54	9.43
275.85	0.038	0.048	1.5	28.13	23.08	9.81
309.29	0.038	0.048	1.6	30.40	23.62	10.17
312.74	0.038	0.048	1.61	30.63	23.67	10.21
316.20	0.038	0.048	1.62	30.86	23.72	10.25
319.68	0.038	0.048	1.63	31.09	23.78	10.28
323.18	0.038	0.048	1.64	31.32	23.83	10.32
344.56	0.038	0.048	1.7	32.73	24.15	10.53
381.64	0.038	0.048	1.8	35.10	24.69	10.87
461.33	0.038	0.048	2	40.00	25.77	11.53
503.95	0.038	0.048	2.1	42.53	26.31	11.85
548.45	0.038	0.048	2.2	45.10	26.85	12.16

PROTECO LANDFILL
REACH "L" DRAINAGE CALCULATIONS
SLOPE = 7.1%, n=0.038

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter (ft)	Velocity (fps)
3.36	0.038	0.071	0.1	1.53	15.54	2.21
10.75	0.038	0.071	0.2	3.10	16.08	3.47
21.26	0.038	0.071	0.3	4.73	16.62	4.50
34.54	0.038	0.071	0.4	6.40	17.15	5.40
50.40	0.038	0.071	0.5	8.13	17.69	6.20
68.71	0.038	0.071	0.6	9.90	18.23	6.94
89.38	0.038	0.071	0.7	11.73	18.77	7.62
112.36	0.038	0.071	0.8	13.60	19.31	8.26
137.60	0.038	0.071	0.9	15.53	19.85	8.86
165.07	0.038	0.071	1	17.50	20.39	9.43
194.76	0.038	0.071	1.1	19.53	20.92	9.97
226.65	0.038	0.071	1.2	21.60	21.46	10.49
260.73	0.038	0.071	1.3	23.73	22.00	10.99
297.01	0.038	0.071	1.4	25.90	22.54	11.47
304.53	0.038	0.071	1.42	26.34	22.65	11.56
308.32	0.038	0.071	1.43	26.56	22.70	11.61
312.14	0.038	0.071	1.44	26.78	22.75	11.65
315.97	0.038	0.071	1.45	27.01	22.81	11.70
335.49	0.038	0.071	1.5	28.13	23.08	11.93
376.17	0.038	0.071	1.6	30.40	23.62	12.37
419.05	0.038	0.071	1.7	32.73	24.15	12.81
464.16	0.038	0.071	1.8	35.10	24.69	13.22
511.50	0.038	0.071	1.9	37.53	25.23	13.63
561.08	0.038	0.071	2	40.00	25.77	14.03
612.91	0.038	0.071	2.1	42.53	26.31	14.41
667.03	0.038	0.071	2.2	45.10	26.85	14.79

PROTECO LANDFILL
REACH "L" DRAINAGE CALCULATIONS
SLOPE = 4.8%, n=0.0395

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter (ft)	Velocity (fps)
2.66	0.0395	0.048	0.1	1.53	15.54	1.74
8.50	0.0395	0.048	0.2	3.10	16.08	2.74
16.82	0.0395	0.048	0.3	4.73	16.62	3.56
27.32	0.0395	0.048	0.4	6.40	17.15	4.27
39.87	0.0395	0.048	0.5	8.13	17.69	4.91
54.35	0.0395	0.048	0.6	9.90	18.23	5.49
70.70	0.0395	0.048	0.7	11.73	18.77	6.03
88.87	0.0395	0.048	0.8	13.60	19.31	6.53
108.84	0.0395	0.048	0.9	15.53	19.85	7.01
130.57	0.0395	0.048	1	17.50	20.39	7.46
154.05	0.0395	0.048	1.1	19.53	20.92	7.89
179.28	0.0395	0.048	1.2	21.60	21.46	8.30
206.24	0.0395	0.048	1.3	23.73	22.00	8.69
234.94	0.0395	0.048	1.4	25.90	22.54	9.07
265.37	0.0395	0.048	1.5	28.13	23.08	9.44
297.55	0.0395	0.048	1.6	30.40	23.62	9.79
307.54	0.0395	0.048	1.63	31.09	23.78	9.89
310.91	0.0395	0.048	1.64	31.32	23.83	9.93
314.29	0.0395	0.048	1.65	31.56	23.89	9.96
317.69	0.0395	0.048	1.66	31.79	23.94	9.99
331.47	0.0395	0.048	1.7	32.73	24.15	10.13
367.15	0.0395	0.048	1.8	35.10	24.69	10.46
443.81	0.0395	0.048	2	40.00	25.77	11.10
484.82	0.0395	0.048	2.1	42.53	26.31	11.40
527.62	0.0395	0.048	2.2	45.10	26.85	11.70

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO		Location Penuelas, P.R		Subject Drainage Swales	
Preparer's Initials JAL	Date 9/20/94	Reviewer's Initials MLP		Date 9/21/94	Approver's Initials	Date

Check spreadsheet for $S = 4.8\ \%$, $n = 0.0395$, $d = 1.64$

$A = 15(d) + 2.5(d)(d)$

$A = 15(1.64) + 2.5(1.64)(1.64)$

$A = 24.6 + 6.72 = 31.32$

$P = 15 + (2)\sqrt{d^2 + (2.5d)^2}$

$P = 15 + (2)\sqrt{(1.64)^2 + (2.5(1.64))^2}$

$P = 23.83$

$Q = 31.32\left(\frac{1.49}{0.0395}\right)\left(\frac{31.32}{23.83}\right)^{2/3}(0.048)^{1/2}$

$Q = 31.32(37.72)\ (1.20)\ (0.22)$

$Q = 310.86$ cfs ok

Check spreadsheet for $S = 7.1\ \%$, $n = 0.0395$, $d = 1.47'$

$A = 15(d) + 2.5(d)(d)$

$A = 15(1.47) + 2.5(1.47)(1.47)$

$A = 22.05 + 5.4 = 27.45$ ok

$P = 15 + (2)\sqrt{(1.47)^2 + (2.5(1.47))^2}$

$P = 15 + 7.92 = 22.92$

$Q = 27.45\left(\frac{1.49}{0.0395}\right)\left(\frac{27.45}{22.92}\right)^{2/3}(0.071)^{1/2}$

$Q = 27.45(37.72)(1.13)(0.27)$

$Q = 311.34$ ok

Proj. No. 16139	Client PROTECO	Location Penuelas, Puerto Rico	Subject Drainage Pipe Calculations
Preparer's Initials JEB	Date 9-26-94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

Drainage Pipe Calculations

Pipe Run "A"

Pipe run a is located on the "Overall Final Grading Plan" Sheet C-8

Total Flowrate (Q) that pipe will receive is based on the 100 year 1 hour duration storm event - (See drainage calculations)

Pipe Run "A" will receive drainage areas/reaches, "F", "G", "H", "I", "Existing drainage ditch B", and the "Detention Basin"

Pipe Run "A" Will Also receive a portion of drainage area "Overland Flow No. 2". The drainage area and corresponding flow rate Q for this area (OL2) are as follows:

On-Site Area (OL2) = 23.12 in² (on a 1"=100' scale)
= 231,200 ft²
= 5.31 Acres

Off-site Area (OL2) = 0.35 in² (on a 1"=1,667 scale)
= 972,611 ft²
= 22.33 Acres

Total Area (OL2) = 27.64 Acres

Using the Rational Method (Q = CIA); ⇒

Weighted C = 19% is a C value of 0.43 and 81% is a C value of 0.31 - (See sheet one of "Drainage Calculations")

$C = (0.19)(0.43) + (0.81)(0.31)$
$C = 0.33$

$Q_m = (0.33)(5.25\text{ in/hr})(27.64\text{ Acres})$
$Q_m = 47.9\text{ cfs}$

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Penuelas, Puerto Rico	Subject Drainage Pipe Calculations
Preparer's Initials JEB	Date 9-26-94	Reviewer's Initials MLP	Date 9-29-94
Approver's Initials	Date		

Total flowrate (Q_{TOT}) to Run "A" is:

$Q_{TOT} = Q_{OLZ} + Q_F + Q_G + Q_H + Q_I + Q_{EB} + Q_{DB}$ (for flowrates for drainage area "F", "G", "H", "I", "Existing Drainage Ditch B" and the "Detention Basin" see "Drainage Calculations"

$Q_{TOT} = (47.9 + 18.0 + 38.0 + 23.6 + 0.3 + 39.0 + 46.9)$ cfs

$Q_{TOT} = 213.7$ cfs

To Calculate the pipe size required to carry Q_T - Manning Equation was used, where:

$Q = (1.49/n)(A)(R)^{2/3}(S)^{0.5}$

where Q = Flowrate in cfs
n = Manning Roughness Coefficient
A = area of pipe (ft^2)
R = hydraulic radius (ft) (for full flowing pipe $R = \frac{\text{Diameter}}{4}$).
S = slope of pipe

For a 42" ϕ Corrugated Metal Pipe (CMP):
$n = 0.022$, Diameter (D) = 3.5', S = 5.0% ⇒

$Q = \left(\frac{1.49}{0.022}\right)\left(\frac{\pi(3.5')^2}{4}\right)\left(\frac{3.5}{4}\right)^{2/3}(0.05)^{0.5}$

$Q = 133.2$ cfs

Using two - 42" ϕ CMP - side by side and at a slope of 5% will provide a total Q of:

$Q = 2(133.2 \text{ cfs})$

$Q_{TOT} = 266.4$ cfs > 213.7 cfs ∴ O.K

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject	
16139	PROTECO	Penuelas Puerto Rico	Drainage Pipe Calculations	
Preparer's Initials JEB	Date 9-27-94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

Must determine headwater depth to determine if headwater (HW) elevation is sufficient

Using the "Inlet Control Chart For Circular CMP Culverts" (with a headwall) (Attached)
For a 42" ϕ CMP with Q = 106.9 cfs (half of design Q)

$\frac{HW}{D} = 2.0 \Rightarrow HW = 2.0(3.5')$
$HW = 7.0'$

The proposed inlet elevation of Pipe Run "A" is 317.75'
The headwater elevation (HE) required is
HE = 317.75' + 7.0'
HE = 324.75

The proposed ground surface elevation above the inlet is:
325.00 > 324.75 ∴ OK

Pipe Run "B"
Pipe Run "B" is shown on the "Sediment Basin" Plan
Sheet C-11

Total Flowrate Q that pipe will receive is based on a 100 year, 1 hour duration storm event (see Drainage Calculations)

Pipe Run "B" will receive drainage areas/reaches "A", "B", "C", "D", "E", and "Overland Existing Flow No. 1 (OE1).

$Q_{PRB} = Q_A + Q_B + Q_C + Q_D + Q_E + Q_{OE1}$
$Q_{PRB} = (5.6 + 19.1 + 20.3 + 3.6 + 1.0 + 65.0)$ cfs
$Q_{PRB} = 114.6$ cfs

Pipe Size Required to carry this flowrate:
For a 36" ϕ CMP, n = 0.022, D = 3.0', S = 2.3%

$Q = \left(\frac{1.49}{0.022}\right)\left(\frac{\pi(3)^2}{4}\right)\left(\frac{3}{4}\right)^{2/3}(0.023)^{0.5}$

$Q = 59.9$ cfs

Using two - 36" ϕ CMP's - side by side and at a slope of 2.3% will provide a total Q of: $Q = 2(59.9 \text{ cfs})$
$Q_{TOT} = 119.8$ cfs > 114.6 cfs ∴ OK

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Penuelas, Puerto Rico	Subject Drainage Pipe Calculations
Preparer's Initials JEB	Date 9-28-94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

Must Determine headwater depth to determine if headwater elevation is sufficient

Using the "Inlet Control Chart for Circular CMP Culverts" (Attached) For D = 36" ø CMP (with a headwall) and Q = 57.3 cfs (half of design Q)

$\frac{HW}{D} = 1.55 \Rightarrow HW = (1.55)(3.0)$

$\underline{HW = 4.65'}$

The Proposed inlet elevation of Pipe Run "B" is 267.20'
The Headwater Elevation (HE) required is:

HE = 267.20' + 4.65'
$\underline{HE = 271.85'}$

The proposed ground surface elevation above the inlet is is $\underline{272.00' > 271.85' \therefore OK.}$

Pipe Run "C"

Pipe Run "C" is shown on the "Sediment Basin Plan Sheet C-11

Total Flowrate that Pipe Run "C" will receive will be that exiting Pipe Run "B" (Q = 114.6 cfs) plus a small drainage area adjacent to and surrounding the PROTECO office Building. The Flowrate for this area is as follows:

A = 6.37 in² (on a 1" = 100' scale map)
A = 63,700 ft²
A = 1.46 Acres

C = 0.43 (see sheet 1 of drainage calculations)
I = 5.25 in/hr (See sheet 2 of calculations)
Q = (0.43)(5.25 in/hr)(1.46 Acres)
$\underline{Q = 3.3 \text{ cfs}}$

Total flowrate to Pipe Run "C" is:

$Q_{PRC} = 114.6 \text{ cfs} + 3.3 \text{ cfs}$
$\underline{Q_{PRC} = 117.9 \text{ cfs}}$

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas Puerto Rico	Drainage Pipe Calculations

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-28-94	MLP	9-29-94		

Pipe size required to carry this flow rate:

For a 36" ϕ CMP, $n = 0.022$, $D = 3.0'$, $S = 3.0\%$

$$Q = \left(\frac{1.49}{0.022}\right)\left(\frac{\pi(3)^2}{4}\right)\left(\frac{3}{4}\right)^{2/3}(0.03)^{0.5}$$

$Q = 68.4$ cfs

Using two - 36" ϕ CMP's - side by side and at a slope of 3.0% will provide a total Q of: $Q = 2(68.4\text{ cfs})$

$Q = 136.8 \text{ cfs} > 117.9 \text{ cfs} \therefore$ OK

Must Determine headwater depth to determine if headwater elevation is sufficient.

Using the "Inlet Control Chart for Circular CMP Culverts" (Attached) For a 36" ϕ CMP (with headwall) and $Q = 59.0$ cfs (half of design Q)

$\frac{HW}{D} = 1.6 \Rightarrow HW = (1.6)(3.0)$

$HW = 4.8'$

The proposed inlet elevation of pipe run "C" is 263.94'

The headwater elevation (HE) required is:

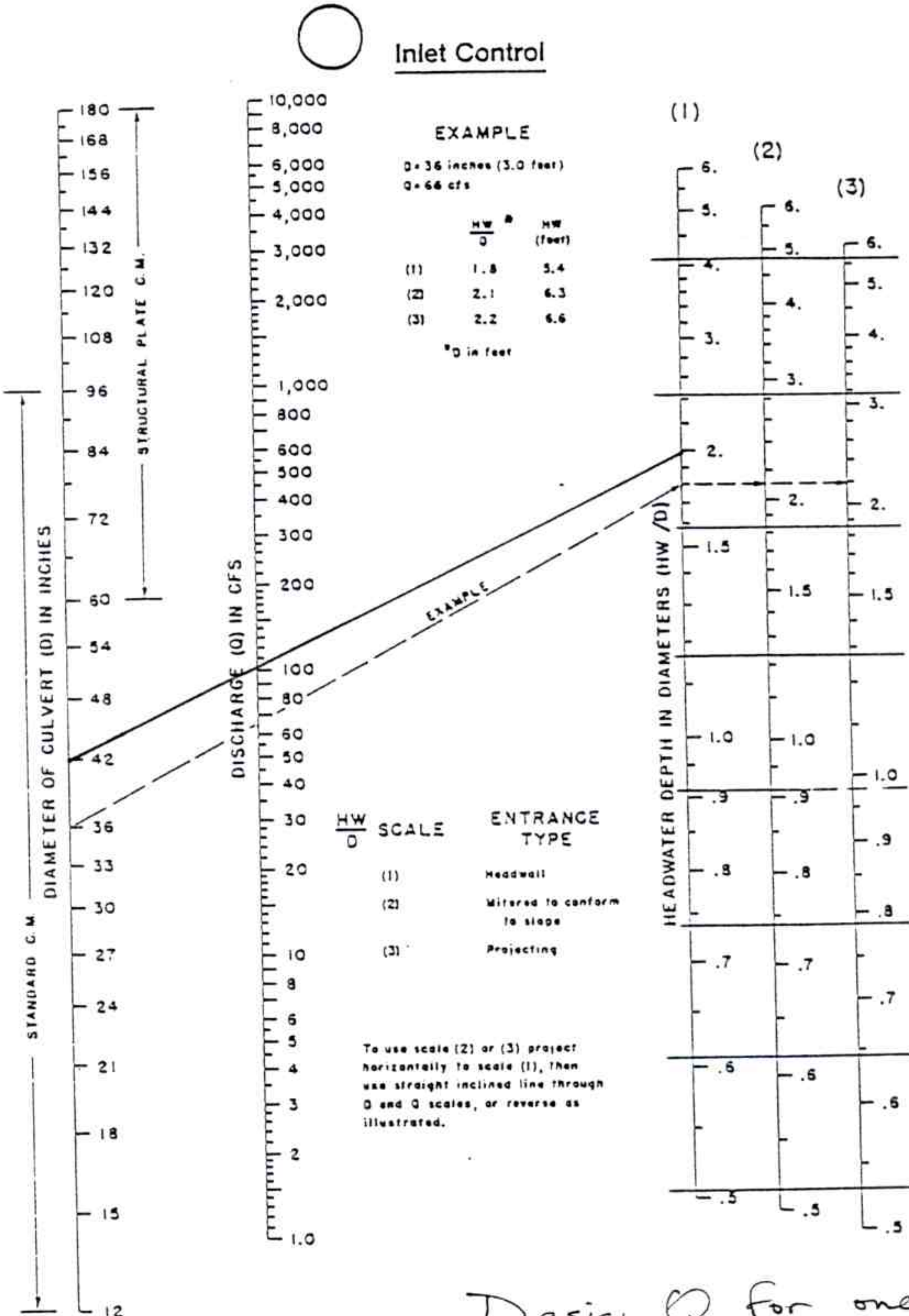
$HE = 263.94' + 4.8'$

$HE = 268.74'$

The proposed surface elevation above the inlet is

$270.00' > 268.74' \therefore$ OK

Pipe Run "A"



Design Q for one 42"φ CMP = 106.9 cfs ∴

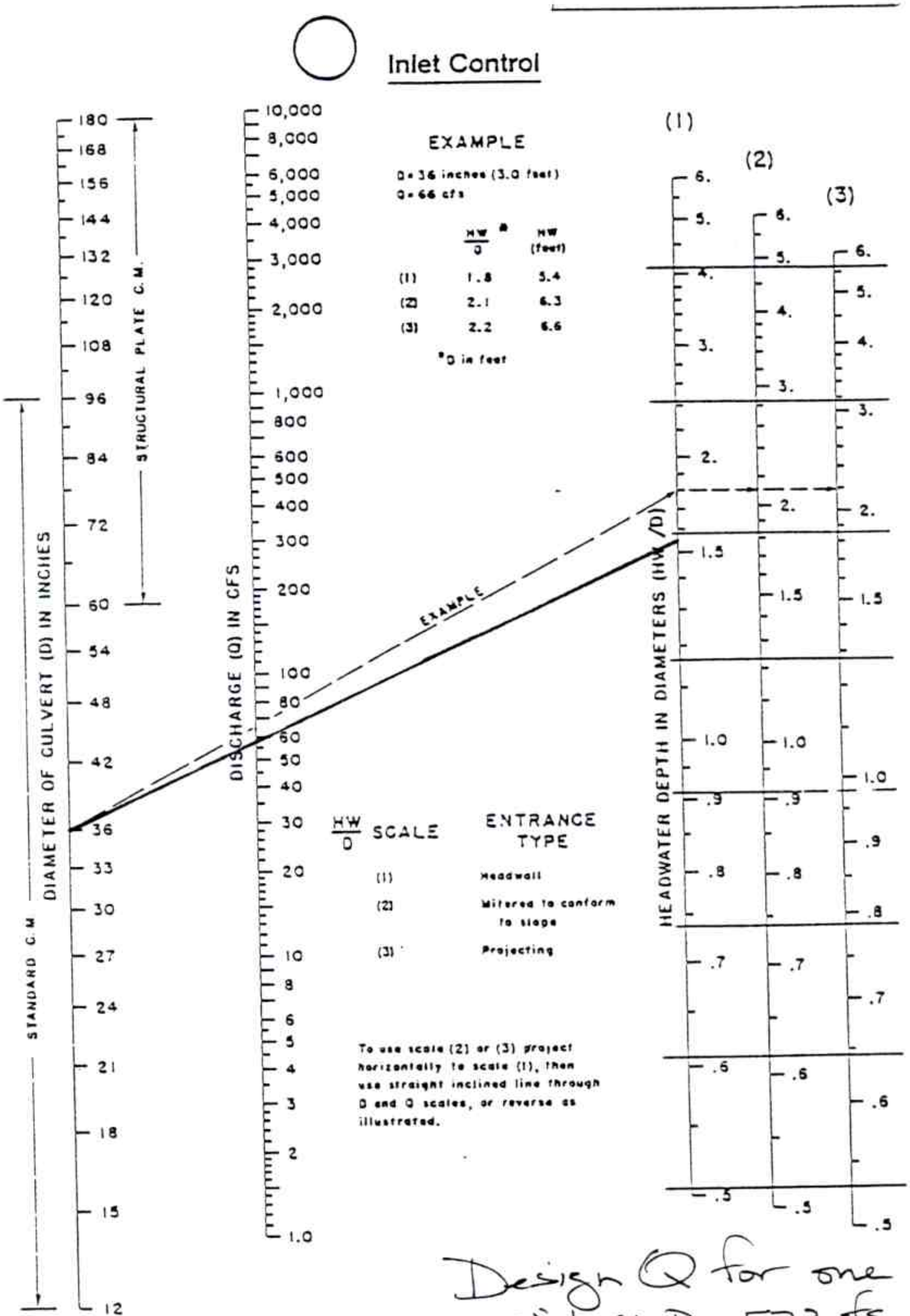
$\frac{HW}{D} = 2.0 \Rightarrow HW = (2.0)(3.5')$

$HW = 7.0'$

Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 5-7
Inlet Control Chart for Circular CMP Culverts

PIPE RUN "B"



Design Q for one
36" ϕ CMP= 57.3 cfs

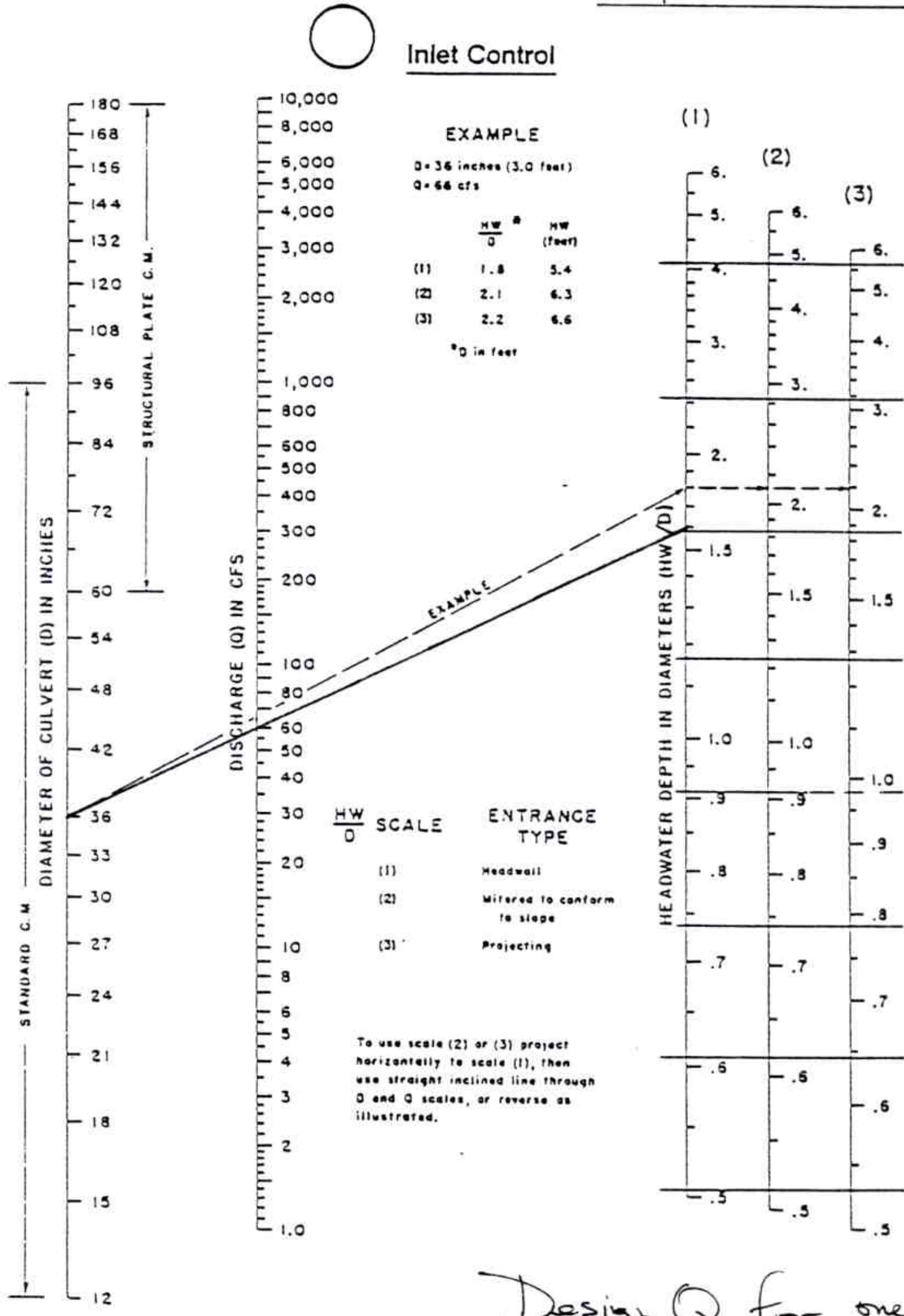
$\frac{HW}{D} = 1.55 \Rightarrow HW = (1.55)(3.0')$

HW = 4.65'

Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 5-7
Inlet Control Chart for Circular CMP Culverts

Pipe Run "C"



Design Q for one 36" ϕ CMP is 59.0 cfs

$\frac{HW}{D} = 1.6 \Rightarrow HW = (1.6)(3.0')$

HW = 4.8'

Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 5-7
Inlet Control Chart for Circular CMP Culverts

APPENDIX D

RETENTION AND SEDIMENT BASINS
DESIGN CALCULATIONS

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Page 1 of 7

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Retention Basin

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-28-94	MLP	9-29-94		

The Retention basin at the site is existing. During construction of CWP's, the height of the existing retention basin will be increased for additional storage capacity.

Using a polar planimeter, the surface area for contour lines 344, 346, 348, 350, 352, 354, 356, 358, 360, 362, & 362.5 were obtained (see attach earthwork quantity sheet) to determine the Volume of the retention basin.

Based on these areas and a contour interval of 2 feet the Volume of the retention basin is:

$\underline{\underline{V_{RB} = 667{,}917\ ft^3}}$

The retention basin receives the following drainage areas; Existing Ditch B (Q_{EDB}), Reach "F" (Q_F), Reach "G" (Q_G), Reach "H" (Q_H), Retention Basin (Q_{RB}), and Reach I (Q_I)

Total inflow to retention is:

$Q_{RBT} = Q_{EDB} + Q_F + Q_G + Q_H + Q_I + Q_{RB}$

$Q_{RBT} = (39.0 + 18.0 + 38.0 + 23.6 + 0.3 + 46.9)$ cfs (for calculation of these values see "Drainage Calculations")

$Q_{RBT} = \underline{165.8\ CFS}$

Capacity of retention basin is determined by the following:

$$\frac{V_{RB}}{Q_{RBT}} = \frac{667{,}917\ ft^3}{165.8\ ft^3/sec}$$

$= 4028.5$ sec
$= 67.1$ min

The design storm is based on a 100 year storm with a 1 hour (60 min) duration

67.1 min > 60.0 min ∴ Retention basin can retain a rainfall event > 100 year, 1 hour duration storm event.

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Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO		Location Peñuelas, Puerto Rico		Subject Retention Basin	
Preparer's Initials JEB	Date 9-29-94	Reviewer's Initials MLP		Date 9-29-94	Approver's Initials	Date

The retention basin will also function as a sediment basin. Since the purpose of the retention basin is to retain/hold water for facility use, principal spillway will not be provided, however, (an overflow) emergency spillway is provided.

Need to determine recommended surface area for optimum settlement of the soil particles.
$A = 1.2(Q_p/V_s)$ (see calculations for Sediment Basin)

Q_p = Peak Inflow based on a 10 year, 24 hour storm event
I for 10 year, 24 hr event = 0.40 in/hr (see Sediment Basin Calcs)
Using this I value Q_p is

$Q_{BUS} = (0.32)(0.9\ in/hr)(23.19\ Acres) = 6.7\ cfs$
$Q_F = (0.32)(0.9\ in/hr)(10.69\ Acres) = 3.1\ cfs$
$Q_G = (0.31)(0.9\ in/hr)(23.33\ Acres) = 6.5\ cfs$
$Q_H = (0.32)(0.9\ in/hr)(14.03\ Ac) = 4.0\ cfs$
$Q_I = (0.43)(0.9\ in/hr)(0.12\ Ac) = 0.1\ cfs$
$Q_{2B} = (0.33)(0.9\ in/hr)(23.00\ Ac) = 7.0\ cfs$
$Q_p = 29.4\ cfs$

(For Coefficient and Areas used in the calculations above see Runoff calculations)

$A = 1.2\left(\frac{29.4\ cfs}{9.6 \times 10^{-4}\ ft/s}\right)$
$A = 35{,}500\ ft^2$

Using a polar planimeter, the surface area of the basin at the crest of the spillway (362.5') is 66,304 ft²
$66{,}304\ ft^2 > 35{,}500\ ft^2$ ∴ OK

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Retention Basin

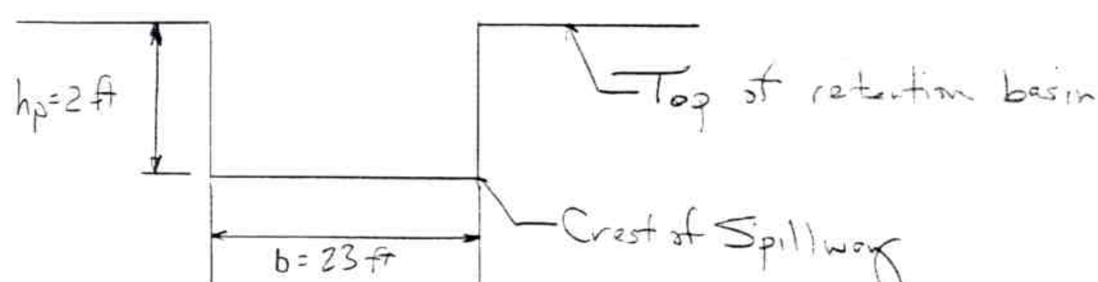
Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-29-94	MLP	9-29-94		

Spillway Design

The spillway is designed to carry Q_{BST} = 165.8 cfs
Using table 6-15.5 (Attached) from the Manual for Erosion and Sediment Control in Georgia (see references) the spillway will have the following dimensions:
23 feet wide by 2 feet deep by 10 feet long (the width of the top of the detention basin

At 23 feet width and 2 foot depth (h_p) the spillway will discharge 171 cfs > 165.8 cfs ∴ OK

Spillway Crest Elev - 362.00'
Spillway Exit Elev - 361.50'



Sediment Depth Storage
Sediment yield to retention basin is based on the universal soil loss equation
A = (R)(K)(LS)(C)(P) - See Sediment Basin Calcs.

R = 200 - See sediment basin calcs.

K = 0.45 - See sediment basin calcs

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Retention Basin

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-29-94	MLP	9-29-94		

Need to Calculate LS

LS is based on the largest anticipated disturbed areas adjacent to and surrounding Waste Units No 2 & 3

Using a polar planimeter, the largest anticipated disturbed area is estimated to be:

$A = 6.82 \text{ in}^2$ (on a 1"=100' scale)

$A = 68{,}200 \text{ ft}^2$

$A = 1.6$ Acres

LS will be based on the average slope being > 5.0%

Based on Wischmeirs Empirical Equation (see Sediment Basin Calcs)

The Longest distance traveled for slope over 5% is L = 280'

$$\text{Slope } S = \frac{\Delta H}{\Delta L} = \frac{406' - 350'}{280'}$$

$S = 20\%$

For Slope over 5% (from Erosion & Sed. Control Handbook)

$m = 0.5$

$$LS = \left[\left(\frac{(65.41)(20)^2}{20^2 + 10{,}000}\right) + \left(\frac{(4.56)(20)}{\sqrt{20^2 + 10{,}000}}\right) + 0.065\right]\left[\frac{280'}{72.5}\right]^{.5}$$

$LS = 6.8$

Sediment Loss A will be calculated for two separate scenarios, 1) based on no ground cover during construction 2) based on final grown cover during postclosure

1) No Ground Cover

1) From the "Manual for Sediment and Erosion Control in Georgia"

C = 0.45 (for 0% ground cover with no appreciable canopy)

P = 0.9 ∴

$A = (200)(0.45)(6.8)(0.45)(0.9)$

A = 247.9 say 248 tons/Acre/yr

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Ponce, Puerto Rico	Subject Retention Basin
Preparer's Initials JEB	Date 9-29-94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

1) No ground cover (cont)

A = 247.9 ton/acre/year

Based on 1.6 disturbed acres

A = 396.6 tons of soil/year

Based on Soil density of 1.5 tons/cu. yd

A = 264.4 cu yd of soil/year

It is recommended that sediment should be removed when capacity of basin is 1/3 full (see Sediment Basin Calculations)

Volume of retention basin is (see Earth work quantities sheet)

$V = 667{,}917\ ft^3$

$V = 24{,}738\ yd^3$

One third of total V is

$V_{1/3} = 8{,}246\ yd^3$

Thus sediment should be removed from the basin as follows

$$\frac{8{,}246\ yd^3}{264.4\ yd^3/yr} = 31.2\ \text{years}$$

Based on these calculations, the retention basin will not be required to be cleaned out during the 30 year post closure period.

Based on the calculations above, no further evaluation of sediment quantities will be required since the retention basin will not be required to be cleaned out during post closure activities, based on No ground cover. However, PROTECO may at their discretion, elect to clean out the retention basin based on the necessity for additional water storage capacity.

For reference - see Sediment Basin Calculations

FOR PROTECO-Retention Basin

W.O.
DATE 9-28-94
SHEET ___ OF ___

[redacted]

CONTOUR	CUT SQ. IN.	CUT SQ. FT.	CUT CUBIC ~~YARDS~~ FEET	FILL SQ. IN.	FILL SQ. FT.	FILL CUBIC YARDS
344	0.0	0.0				
			15,152			
46	9.47	15,152				
			36,305			
48	13.22	21,152				
			51,872			
350	19.20	30,720				
			65,264			
352	21.59	34,544				
			73,776			
54	24.52	39,232				
			83,232			
56	27.50	44,000				
			93,072			
358	30.67	49,072				
			103,440			
60	33.98	54,368				
			114,256			
62	37.43	59,888				
			31,548			
362.5	41.44	66,304				
		Total	667,917 ft3			

SUB TOTAL

SUB TOTAL

Cut
Shrinkage

TOTAL FILL

MLP 9-29-94

Retention Basin Sheet 7 of 7

DESIGN DATA FOR EARTH SPILLWAYS

STAGE (Hp) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET 8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.8	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	3.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3

DATA TO RIGHT OF HEAVY VERTICAL LINES SHOULD BE USED WITH CAUTION, AS THE RESULTING SECTIONS WILL BE EITHER POORLY PROPORTIONED, OR HAVE VELOCITIES IN EXCESS OF 6 FEET PER SECOND.

Source: USDA-SCS

Table 6-15.5

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16137	Client PROTECO	Location Peñuelas, Puerto Rico	Subject Sediment Basin
Preparer's Initials JEB	Date 9-14-94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

Design Calculations for Sediment Basin

The design volume for the sediment basin is based on "on-site" drainage areas for Reaches A, B, C, D, E, and Existing Overland Flow No 1. These drainage areas are adjacent to and surrounding Waste Units No 9, 10, 11, 12, 13, 16, and 17). This area is being used because it will have the largest disturbed area(s). The "off-site" drainage area outside of these design areas are heavily vegetated and will not be disturbed during construction activities. The drainage areas adjacent to and surrounding Waste Units 1, 2, 3, and 5 are not being considered in the design calculations for the [illegible] sediment basin, as the surface water run-off from these areas discharges directly into the existing detention basin.

The design drainage area for the sediment basin is 22.16 acres

Design volume of the sediment basin is based on providing 67 cu yds. of storage capacity per acre of disturbed area draining into the basin. (67 cu. yds. is equivalent to ½ inch of sediment per acre of basin drainage area)

Design Volume of Sediment basin shall be:
(22.16 acres)(67 cu yds/acre) = 1,485 cu.yds.
= 40,095 cu.ft.

Peak Inflow (Q)

Using the Rational Method Q = CIA

The peak inflow (Q) sizing the sediment basin (required surface area) is based on 10 year storm event with a 24 hour duration (as required in the EPA Technology Transfer Seminar Publication - "Erosion and Sediment Control - Surface Mining in the Eastern U.S. - Design")

The intensity (I) for a 10 year, 24 hour storm event for Peñuelas, Puerto Rico (based on Technical Paper No. 42) is:

I = 9.5 inches / 24 hours
I = 0.40 inches / hour

COMPUTATION SHEET

OHM Corporation

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Sediment Basin

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-16-94	MLP	9-29-94		

The drainage area (A) is based on the "off-site" and "on-site" drainage areas for Reaches A, B, C, D, and E, existing overland flow no. 1, and for off-site drainage area # 1.

$A = 95.29$ acres

The runoff coefficient (C) is based on a weighted value; based on existing and proposed site conditions, approximately 23% of the drainage area will be grass, lawn, pasture land with heavy soil at a slope ≤ 7% (C=0.27) and that approximately 77% of the drainage area will be woodlands with heavy vegetation with heavy soil at a slope > 7% (C=0.43):

$C = (0.23)(0.27) + (0.77)(0.43)$

$C = 0.39$

Peak inflow $(Q_P) = CIA$

$Q_P = (0.39)(0.40\,in/hr)(95.29\,Acres)$

$Q_P = 14.9\,cfs$

The recommended surface area for the sediment basin is based on Q_P and V_S - the settling velocity for soil particles. For the site, the average soil particles will be 0.02 mm (medium silt) the V_S 9.6×10^{-4} ft/sec. Using a 1.2 factor of safety, the required surface area for the sediment basin is:

$A = 1.2\,(Q_P / V_S)$

$A = 1.2\,(14.9\,cfs / 9.6 \times 10^{-4}\,ft/sec)$

$A = 18{,}625\,ft^2$

$A = 0.43$ Acres

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Peñuelas Puerto Rico	Sediment Basin

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-14-94	MLP	9-29-94		

Design Specifications for Sediment Basin and Principle and Emergency Spillway

Crest of Principle Spillway - Elevation - 261.00
Crest of Emergency Spillway Elevation - 262.00
Top of dentention Basin - Elevation - 264.00
Bottom of dentention basin - Elevation - 249.00

Using a polar planimeter the surface area at the elevation of the principle spillway (260.00), on a 1" = 40' scale, is - 0.46 acres (20,064 ft^2) which is > the required 0.43 acres (18,625 ft^2) therefor, O.K.

Using a polar planimeter, on a 1" = 40' scale, to obtain the surface area at elevations 250, 252, 254, 256, 258, and 260, the volume of the sediment basin is - 105,896 ft^3 which is >> than the required volume of 40,095 ft^3. The governing factor for sizing the sediment basin is the required surface, thus, is the reasoning for the excess volume provided in the sediment basin design. The worksheets used for calculating the volume of the sediment basin are attached.

Stand Pipe and Discharge Pipe Sizing

Discharge Pipe (Horizontal)
Discharge pipe must be sized to carry 14.9 cfs
Invert elevation into pipe will be set at elevation 248.00
Invert elevation out of pipe will be set at elevation 235.90
124 L.F. of discharge pipe will be required.
Slope of pipe will be set at 9.8%

Try a 12" ϕ CMP (Minimum discharge pipe diameter (ϕ))
Using Manning equation: $Q = (1.49/n)(A)(R_H)^{2/3}(S)^{0.5}$
The Hydraulic radius (R_H) for a full flowing pipe is D/4

$$Q = \left(\frac{1.49}{0.022}\right)\left(\frac{\pi (1)^2}{4}\right)\left(\frac{1}{4}\right)^{2/3}(0.098)^{.5}$$

$Q = 6.6$ cfs < 14.9 cfs ∴ must try larger pipe

OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Sediment Basin

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-15-94	MLP	9-29-94		

Try a 15" ϕ CMP

$$Q = \left(\frac{1.49}{0.022}\right)\left(\frac{\pi(1.25)^2}{4}\right)\left(\frac{1.25}{4}\right)^{2/3}(0.098)^{0.5}$$

Q = 11.9 cfs < 14.9 cfs ∴ must try larger pipe

Try an 18" ϕ CMP

$$Q = \left(\frac{1.49}{0.022}\right)\left(\frac{\pi(1.5)^2}{4}\right)\left(\frac{1.5}{4}\right)^{2/3}(0.098)^{0.5}$$

Q = 19.4 cfs > 14.9 cfs ∴ OK

<u>Use a 18" ϕ CMP for the discharge pipe</u>

Stand Pipe (Vertical Riser)
Using Table [illegible] (attached) [illegible] the "Manual For [illegible] [illegible] Control [illegible]"
A 2[illegible]" ϕ CMP Stand Pipe will be required for an 18" ϕ [illegible] discharge pipe. Using an "h" (distance between crest of [illegible] and the crest of the emergency spillway) of 1 foot, the Stand Pipe can carry 19.5 cfs > 14.9 cfs ∴ OK

<u>Use a 24" ϕ CMP for the stand pipe</u>

A 36" ϕ Trash rack is required for a 24" ϕ Stand pipe

<u>Emergency Spillway Design</u>
The emergency spillway is designed to carry the difference in flow from a 100 year, 1 hour duration storm event and the 10 year, 24 hour duration storm event.

Q_{100} = CIA - From Technical Paper No 42, the intensity for a 100 year storm, 1 hour duration is
I = 5.25 inches / 1 hour duration
I = 5.25 inches / hour

Q_{100} = (0.39)(5.25 in/hr)(95.29 Acres)
Q_{100} = (195.1 cfs)

Q_{10} = 14.9 cfs (from previous calculations)

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Sediment Basin

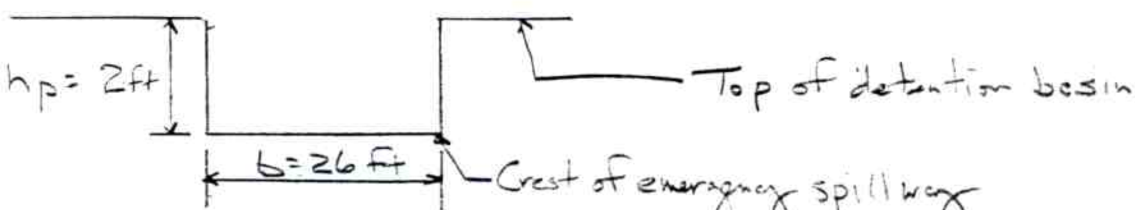
Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-16-94	MLP	9-29-94		

Thus, the emergency spillway must carry/discharge difference of flow (Q_E) of Q_{100} and Q_{10}

$$Q_E = Q_{100} - Q_{10}$$
$$Q_E = 195.1\,cfs - 14.9\,cfs$$
$$Q_E = 180.2\,cfs$$

Using table 6-15.5 (attached) from the "Manual for Erosion and Sediment Control in Georgia" the emergency spillway will have the following dimensions:
26 feet wide by 2 feet deep by 10 feet long (the width of the top of the detention basin)

At 26 foot width and 2 ft depth (h_p) the emergency spillway will discharge 193 cfs > 180.2 cfs ∴ OK



<u>Sediment Storage Depth</u>

The sediment yeild to the basin is calculated using the Universal Soil Loss Equation:

$$A = (R)(K)(LS)(C)(P) \text{ where}$$

A = Soil Loss in tons per acre year
R = Rainfall erosion index in 100 ft·tons/acre × in/hr
K = Soil erodibility factor, tons/acre per unit of R
LS = slope length and steepness factor, dimensionless
C = vegetative cover factor, dimensionless
P = erosion control practice factor, dimensionless

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO		Location Penuelas, Puerto Rico		Subject Sediment Basin	
Preparer's Initials JEB	Date 9-23-94	Reviewer's Initials		Date	Approver's Initials	Date

The rainfall erosion [illegible] (R) [illegible] was obtained from the "Caribbean Area Rainfall Values (R values) - Universal Soil Loss Equation" - This sheet is attached. Based on the location of the site;

$R = 200$

Soil Erosion Factor (K)

Based on the sieve analysis of soil samples obtained from the site - see Geotechnical Report - approximately 75% of the soil is silt and approximately 25% of the soil is clay. Using the triangular nomograph (attached) the K value is determined to be:

$K = 0.45$

Length/Slope Factor (LS)

The length/slope factor is be based on the largest anticipated disturbed areas adjacent to and surrounding Waste Units 9, 10, 11, 12, 13, 16, 17. The LS factor will be based on a weighted value.

Using a polar planimeter, the largest anticipated disturbed area is estimated to be

$A = 40.71 \text{ in}^2$ (on a 1"~100' scale)
$A = 407,100 \text{ ft}^2$
$A_{TOT} = 9.35$ Acres

Of this total area approximately 3.0 Acres will have a surface slope between 3.0% and 5.0% (32%)

The remainder of the area will have a surface slope greater than 5.0% (68%)

Proj. No.	Client	Location	Subject	
16139	PROTECO	Penuelas, Puerto Rico	Sediment Basin	
Preparer's Initials JEB	Date 9-25-94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

LS for Slope between 3.0% and 5.0%

Based on Wischmeier's empirical equation (Reference "Erosion and Sediment Control Handbook - See References), the L/S factor can be calculated as follows:

$$LS = \left[\left(\frac{(65.41)(S^2)}{S^2+10{,}000}\right) + \left(\frac{4.56 \times S}{\sqrt{S^2+10{,}000}}\right) + 0.065\right]\left[\left(\frac{L}{72.5}\right)^m\right]$$

where L = slope length in ft
S = slope steepness in %
m = exponent dependent upon slope steepness

For slope between 3.0% & 5.0% the longest distance traveled L = 1,060 feet

The average slope over this distance is:

$$S = \frac{\Delta H}{\Delta L} = \frac{340ft - 300ft}{1{,}060 feet} \Rightarrow S = 3.8\%$$

For a slope between 3.5% and 4.5% (from Erosion and Sediment Control HandBook"
m = 0.4

$$LS_{3-5\%} = \left[\left(\frac{(65.41)(3.8)^2}{3.8+10{,}000}\right) + \left(\frac{(4.56)(3.8)}{\sqrt{(3.8)^2-10{,}000}}\right) + 0.065\right]\left[\left(\frac{1{,}060 ft}{72.5}\right)^{0.4}\right]$$

$LS_{3-5\%} = 0.97$

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Penuelas Puerto Rico		Subject Sediment Basin	
Preparer's Initials JEB	Date 9/25/94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

1) No ground cover during Construction (cont)

A = 154.6 tons of sed / acre / year
Based on disturbed area A_{TOT} = 9.35 Acres
A = (154.6 tons of sed / ~~acre~~ / year)(9.35 ~~Acres~~)
A = 1,445.5 say 1,446 tons of sed / year
Based on soil density of 1.5 tons / cu yd
A = 964 cu yd of sed / year

The "Manual for Erosion and Sediment Control in Georgia" - see references - recommends that the sediment build up should be removed when approximately one-third of the storage volume has been filled.

The volume of the sediment basin has been calculated to be:
V = 105,896 ft³
V = 3,922.1 yd³

One-third of the total volume is:
$V_{1/3}$ = 1,307.4 yd³

With 2,141 cu yd of sed / year and $V_{1/3}$ equal to 1,307.4 yd³, sediment should be removed from the basin as follows
$\frac{964 \text{ cy/year}}{1{,}307.4 \text{ cu yd}}$ = 0.73 times / year say once every 9 months

Thus during construction activities, the sediment basin should be cleaned out once every 9 months.

1,307.4 cu yd correspond to an elevation in the sediment basin of approximately 255 ft.

Thus when sediment in the basin reaches the 255 contour line or 3 feet of depth, the sediment basin shall be cleaned.

COMPUTATION SHEET

OHM Corporation

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 16139	Client PROTECO	Location Penuelas, Puerto Rico	Subject Sediment Basin
Preparer's Initials JEB	Date 9/25/94	Reviewer's Initials MLP	Date 9-29-94	Approver's Initials	Date

2) Post Closure

From the "Manual for Erosion & Sediment Control in Georgia"
C = 0.003 (95-100% ground cover with no appreciable canopy)
P = 0.9

A = (200)(0.45)(4.24)(0.003)(0.9)
A = 1.0 tons of soil / acre / yr
Based on 9.35 acres
A = (1.0 ton of soil / ~~acre~~ / year)(9.35 ~~acres~~)
A = 9.35 tons say 10.0 tons of soil / year
A = 6.7 cu. yd of soil / year

With 6.7 cu. yds of soil / year and $V_{1/3}$ = 1,307.4 yd^3, sediment should be removed from the basin during post closure as follows:

$$\frac{1307.4\ yd^3}{6.7\ yd^3/year} = 195.1\ years$$

Based on these calculations, the sediment basin will not be required to be cleaned out during the 30 year Post Closure Period.

2 27 =C.I.

FOR
PROTECO Sediment Basin

W.O. ______
DATE 9-14-94
SHEET 11 OF 12

[redacted]

TOUR	CUT SQ. IN.	CUT SQ. FT.	CUT CUBIC [redacted] FEET	FILL SQ. IN.	FILL SQ. FT.	FILL CUBIC YARDS
249	0.0	0.0				
			1,944			
.50	2.43	3,888				
			8,992			
.52	3.19	5,104				
			11,808			
254	4.19	6,704				
			19,024			
.56	7.70	12,320				
			28,192			
.58	9.92	15,872				
			35,936			
260	12.54	20,064				
		Total	105,896 ft³			

SUB TOTAL

SUB TOTAL

Cut
Shrinkage

TOTAL FILL

(mkp 9-29-94)

WEIR FLOW (Q) OVER RISER CREST FOR CIRCULAR RISERS WITH TRASH RACK

$$Q = CLh^{3/2}$$

$$Q = 3.1 \times (\pi) \times (D/12) \times h^{3/2}$$

HEAD-h in feet	Riser Diameter (D_r) in Inches: 12	18	24	30	36	48	54	60	HEAD-h in feet
	Flow In Cubic Feet Per Second								
0.1	0.3	0.5	0.6	0.8	0.9	1.2	1.4	1.5	0.1
0.2	0.9	1.3	1.7	2.2	2.6	3.5	3.9	4.4	0.2
0.3	1.6	2.4	3.2	4.0	4.8	6.4	7.2	8.0	0.3
0.4	2.5	3.7	4.9	6.2	7.4	9.9	11.1	12.3	0.4
0.6	4.5	6.8	9.1	11.3	13.6	18.1	20.4	22.6	0.6
0.8		10.5	13.9	17.4	20.9	27.9	31.4	34.8	0.8
1.0			19.5	24.3	29.2	39.0	43.8	48.7	1.0
1.2			25.6	32.0	38.4	51.2	57.6	64.0	1.2
1.4				40.3	48.4	64.5	72.6	80.7	1.4
1.6				49.3	59.1	78.8	88.7	98.6	1.6
1.8					70.6	94.1	105.8	117.6	1.8
2.0					82.6	110.2	124.0	137.7	2.0
2.2						127.1	143.0	158.9	2.2
2.4							162.9	181.0	2.4
2.6							183.7	204.1	2.6
2.8								228.1	2.8
3.0								253.0	3.0

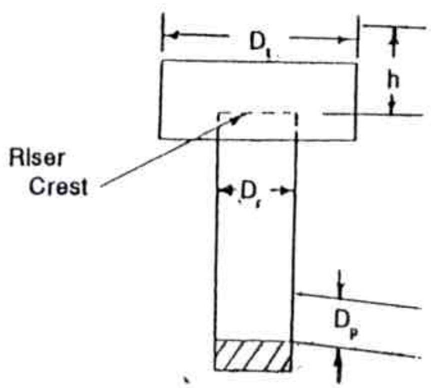
PIPE, RISER, AND TRASH RACK PROPORTIONS

Diameters in Inches

Principal Spillway Pipe D_p	Riser Pipe D_r	Trash Rack D_t
8	12	24
12	18	30
15	21	30
18	24	36
24	30	42
30	36	54
36	48	66
42	54	72
48	60	84

Table 6-15.3 - Design Chart For Conduit Pipe, Riser, And Trash Rack Diameters

Table 6-15.2



EXAMPLE:
The peak runoff for a 2 year, 24 hour rain is 32 cfs. Select a pipe size for a head of 12 feet and length of 100 feet. From Table 6-15.1, 38.2 x 0.89 = 34 cfs discharge for a 24 inch diameter pipe.

From Table 6-15.3, the diameter of the riser is 30 inches and the trash rack is 42 inches.

From Table 6-15.2, 1.2 foot of head (h) above the crest of the riser is required to discharge 32 cfs.

NOTE: h = minimum distance between the crest of the riser and the crest of the emergency spillway.

Sheet 12 of 17

DESIGN DATA FOR EARTH SPILLWAYS

STAGE (Hp) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.9	3.8	3.8	3.8	3.8	3.9	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.6	2.5	2.6	2.5	2.5	2.5
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	3.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3

DATA TO RIGHT OF HEAVY VERTICAL LINES SHOULD BE USED WITH CAUTION, AS THE RESULTING SECTIONS WILL BE EITHER POORLY PROPORTIONED, OR HAVE VELOCITIES IN EXCESS OF 6 FEET PER SECOND.

Source: USDA-SCS

Table 6-15.5

PROTECO LANDFILL
EMERGENCY SPILLWAY DRAINAGE CALCULATI
SLOPE = 40%, n=0.041

Flow (cfs)	n	Slope (%)	Flow Depth (ft)	Area (sq. ft.)	Wetted Perimeter	Velocity (fps)
12.79	0.041	0.4	0.1	2.62	26.45	4.88
40.77	0.041	0.4	0.2	5.28	26.89	7.72
80.37	0.041	0.4	0.3	7.98	27.34	10.07
130.16	0.041	0.4	0.4	10.72	27.79	12.14
170.59	0.041	0.4	0.47	12.66	28.10	13.47
176.72	0.041	0.4	0.48	12.94	28.15	13.66
182.95	0.041	0.4	0.49	13.22	28.19	13.84
189.26	0.041	0.4	0.5	13.50	28.24	14.02
257.08	0.041	0.4	0.6	16.32	28.68	15.75
333.18	0.041	0.4	0.7	19.18	29.13	17.37
417.22	0.041	0.4	0.8	22.08	29.58	18.90
508.93	0.041	0.4	0.9	25.02	30.02	20.34
608.09	0.041	0.4	1	28.00	30.47	21.72
714.53	0.041	0.4	1.1	31.02	30.92	23.03
828.08	0.041	0.4	1.2	34.08	31.37	24.30
948.63	0.041	0.4	1.3	37.18	31.81	25.51

7(mm) = 41.4]

susceptibility of soil particles to
. T xture is the principal factor
p neability also contribute. K

value for a site, but a nomograph
li le. If a recent soil survey for
is rbance is anticipated, the K
on the site can be used.

is the nomograph method. Use
is to determine the percentages
ra ge for each class is listed in
or ter analysis for particle size
nated in the request for analysis.
si intervals, such as every 5 or
na only a small fraction of the

based on the soil exposed during
r ; grading will have K values
it several samples should be
differences in soil texture are
ould be characterized.
er d, the more accurate the K
v iation in soil erodibility, it
different parts of the site and to
ti e areas. A simpler and more
lu obtained by analysis for all
know exactly what soils will be

C Utah office (6), based on the
1) s reproduced in Fig. 5.6. To
two of the particle size percents:
otal sand. Use whole numbers.
of tersection. From that point,
si of the triangle, where the K

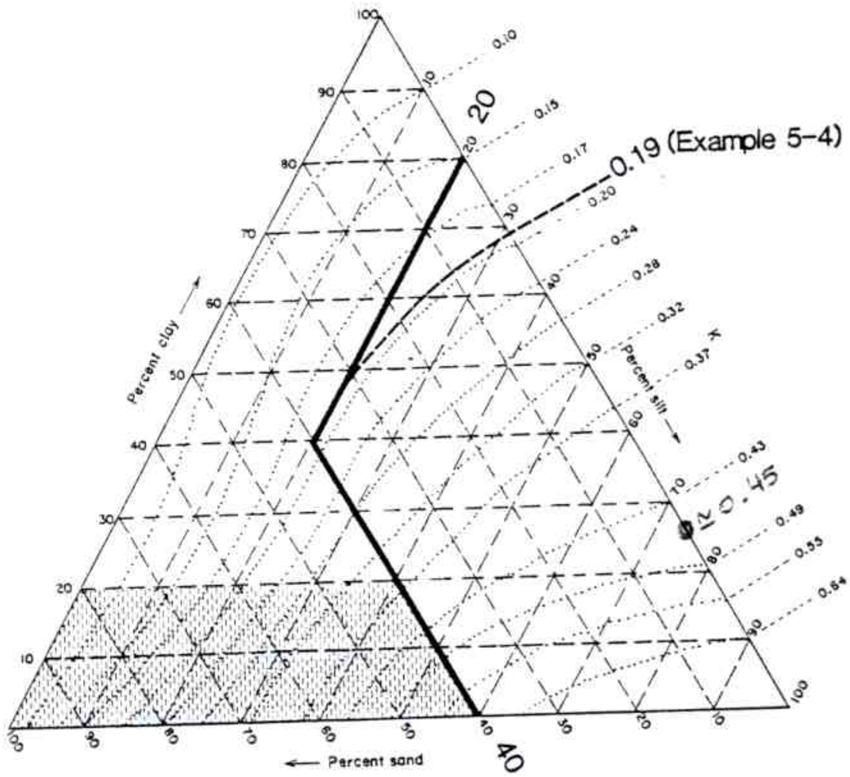


Fig. 5.6 Triangular nomograph for estimating K value. (6) See Table 5.3 for adjustments to K value under certain conditions.

EXAMPLE 5.4

Given: A soil with the following particle size distribution.

Component	Size, mm	Fraction, %
Sand	2.0–0.1	30
Very fine sand	0.1–0.05	10
Silt	0.05–0.002	20
Clay	Less than 0.002	40

Find: Texture and K value.

Solution: Entering Fig. 5.1 with 40 percent total sand and 20 percent silt, the texture is found to be on the border between clay and clay loam. Entering Fig. 5.6 with the same percents (see bold lines), the K value is found to be 0.19.

Table 5.3 describes adjustments to the K factor. Adjustment 1 is a correction for very

OPTIONAL FORM 99 (7-90)

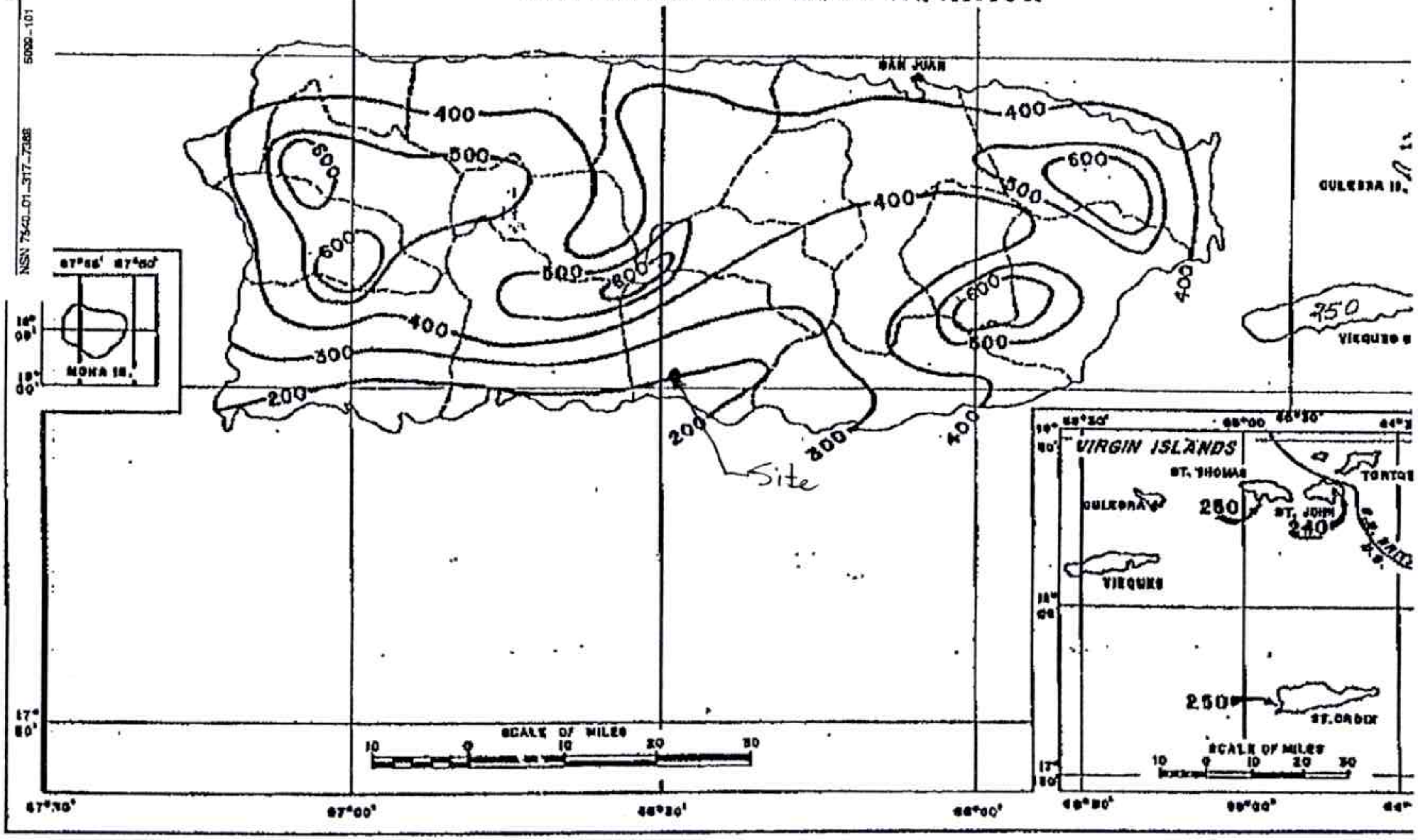
FAX TRANSMITTAL | # of pages ▸ 1

To ED BUCHLI	From BILL MERKEL
Dept./Agency	Phone # 817-334-5242
Fax #	Fax #

NSN 7540-01-317-7368 5099-101 GENERAL SERVICES ADMINISTRATION

8 809 2581754 05/03/93 15:30 SOIL CONSERVATION SER

CARIBBEAN AREA
RAINFALL VALUES (R values)
UNIVERSAL SOIL LOSS EQUATION



OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	PROTECO	Penuelas, Puerto Rico	Sediment Basin

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JEB	9-25-94				

References

1) Steven J. Goldman, Katharine Jackson, Taras A Bursztynsky, PE, "Erosion and Sediment Control Handbook", McGraw-Hill, Inc, U.S.A., 1986

2) Georgia Soil and Water Conservation Commission, "Manual for Erosion and Sediment Control in Georgia" - Third Edition, Athens, 1992

APPENDIX E

SOIL PROPERTY AND INTERFACE DIRECT SHEAR TESTING REPORT

GEOSYNTEC CONSULTANTS

Geomechanics and Environmental Laboratory
5775 Peachtree Dunwoody Road • Suite 10D
Atlanta, Georgia 30342 • USA
Tel. (404) 705-9500 • Fax (404) 705-9300

28 September 1994

Mr. Rene Rodriguez
Vice President, Technical Operations
PROTECO
Carr. 385, 3.5 km
Penuelas, Puerto Rico 00624

Subject: Final Report
Soil Property and Interface Direct Shear Testing
PROTECO Hazardous Waste Units
Penuelas, Puerto Rico

Dear Mr. Rodriguez:

GeoSyntec Consultants (GeoSyntec) is pleased to present the results of the soil property and interface direct shear testing program performed for PROTECO for the PROTECO Hazardous Waste Units project located in Penuelas, Puerto Rico. The testing program was conducted at GeoSyntec's Geomechanics and Environmental Laboratory located in Atlanta, Georgia. This letter report was prepared by Mr. Robert H. Swan, Jr., and Dr. Zehong Yuan, both of GeoSyntec. The report was reviewed by Dr. Gary R. Schmertmann, P.E. (Georgia), also of GeoSyntec, in accordance with the internal peer review policy of the firm.

The testing program was conducted in accordance with the test procedures defined in the 10 June 1994 letter prepared by Mr. Joseph A. Carris of OHM Remediation Services Corporation (OHM) on behalf of PROTECO and PROTECO's Purchase Order No. PO88-2149, issued to GeoSyntec on 3 June 1994. GeoSyntec understands that the purpose of the testing program was to evaluate: (i) the soil properties (i.e., compaction characteristics and dispersive classification) of a site clay soil (OHM sample number P-2); (ii) the hydraulic conductivity of a drainage sand (OHM sample number P-5); (iii) the interface shearing resistance between a National Seal Company (NSC) PN3000SCN geonet and a 20-mil (0.5-mm) thick Staff Industries, Inc. (Staff) smooth polyvinyl chloride (PVC) geomembrane; and (iv) the interface shearing resistance between the site clay and the 20-mil (0.5-mm) thick Staff smooth PVC geomembrane. GeoSyntec also

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Laboratories:
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Boca Raton, FL
Huntington Beach, CA

understands that the sample preparation procedures and testing conditions used in the testing program were selected by OHM on behalf of PROTECO to simulate anticipated field conditions.

The remaining sections of this letter report present: (i) a description of the configuration of the test specimens used in the interface direct shear tests; (ii) the testing procedures used in the soil property, hydraulic conductivity, and interface direct shear tests; and (iii) the test results.

CONFIGURATION OF THE INTERFACE DIRECT SHEAR TEST SPECIMENS

Two interface direct shear test series were conducted to evaluate the interface shearing resistance between: (i) the NSC PN3000SCN geonet and the 20-mil (0.5-mm) thick Staff smooth PVC geomembrane under wet conditions; and (ii) the site clay and the 20-mil (0.5-mm) thick Staff smooth PVC geomembrane under as-placed moisture conditions. Each test series consisted of three interface direct shear tests with each test conducted at a different level of normal stress ranging from 400 to 800 psf (19 to 39 kPa) using a freshly prepared test specimen. Table 1 summarizes the general testing conditions that were used for the two interface direct shear test series. The configurations of the test specimens used in the two interface direct shear test series were as follows:

- *Test Series Number 1:* interface between NSC PN3000SCN geonet and 20-mil (0.5-mm) Staff smooth PVC geomembrane under wet conditions. From top to bottom, each test specimen consisted of:
  - rigid substrate;
  - NSC PN3000SCN geonet;
  - 20-mil (0.5-mm) thick Staff smooth PVC geomembrane; and
  - concrete sand.

- *Test Series Number 2:* interface between site clay and 20-mil (0.5-mm) thick Staff smooth PVC geomembrane under as-placed moisture conditions. From top to bottom, each test specimen consisted of:
  - site clay;
  - 20-mil (0.5-mm) thick Staff smooth PVC geomembrane; and
  - drainage sand.

Bulk samples of the site clay and drainage sand materials used in the testing program were provided to GeoSyntec by PROTECO. OHM arranged for the manufacturer of each of the geosynthetic materials used in the testing program to ship material for testing directly to GeoSyntec. The concrete sand was provided by GeoSyntec to fill in the lower shear box and serve as a bedding layer below each test interface in Test Series 1.

TESTING PROCEDURES

Soil Property Tests

A single-point compaction test on the site clay was conducted in general accordance with the American Society for Testing and Materials (ASTM) Standard Test Method D 698, *"Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb/ft^3 (600 kN-m/m^3))"* in order to confirm the standard Proctor compaction test results of the site clay provided by PROTECO.

A pinhole dispersion test was conducted on a remolded sample of the site clay in general accordance with the ASTM Standard Test Method D 4647, *"Identification and Classification of Dispersive Clay Soils by the Pinhole Test"* in order to evaluate the dispersibility of the site clay. At the request of OHM, the Method A procedure within the test standard was used to conduct the test and report the results.

Hydraulic Conductivity Test

A rigid wall constant head hydraulic conductivity test was performed on a representative specimen of the drainage sand in general accordance with ASTM Standard Test Method D 2434, "*Permeability of Granular Soils (Constant Head)*". The rigid wall hydraulic conductivity test was conducted using a 6-in. (150-mm) diameter rigid-wall permeameter and tap water as the permeant. The test conditions were as follows:

- the test specimen was formed by placing and compacting the moisture-conditioned drainage sand in the 6-in. (150-mm) diameter permeameter in approximately three equal lifts;

- the test specimen was compacted by hand tamping to the reported dry unit weight of 114.7 lb/ft^3 (18.0 kN/m^3) at a moisture content of 10.8 percent; and

- the test specimen was permeated with tap water using a constant head hydraulic gradient of 0.2 throughout testing.

Interface Direct Shear Tests

The interface direct shear tests were performed in general accordance with the ASTM Standard Test Method D 5321, "*Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method*". The tests were conducted in a large direct shear device containing an upper and lower shear box. The upper shear box measures 12 in. by 12 in. (300 mm by 300 mm) in plan and 3 in. (75 mm) in depth. The lower shear box measures 12 in. by 14 in. (300 mm by 350 mm) in plan and 3 in. (75 mm) in depth.

A summary of the test equipment and conditions used to conduct the interface direct shear tests is presented in Table 2. This table indicates the size of the shear box, the initial compaction condition for the soils (i.e., dry unit weight and moisture content), the normal stress applied during consolidation, time for consolidation, the moisture content of the soils at the completion of testing, the normal stress applied to the soil in the upper shear box during shearing, and the horizontal displacement rate for each test.

In all of the interface direct shear tests, fresh test specimens were prepared for each normal stress condition. For each test, the test specimens were set up and tested as described below to achieve the desired moisture condition and to cause shear failure to occur at the desired interface.

- *Test Series Number 1:* fresh specimens of the geonet and geomembrane were trimmed from each bulk sample of material provided by each manufacturer and attached to the upper and lower shear boxes, respectively, with mechanical compression clamps. This set-up caused shearing to occur at the geonet-geomembrane interface. For each test, the geonet and upper surface of the geomembrane were wetted, prior to being sheared, by pouring tap water on top of the geonet specimen and allowing the tap water to drain at the geonet-geomembrane interface.

- *Test Series Number 2:* fresh specimens of the drainage sand were moisture-conditioned and compacted into the lower shear box by hand tamping to the reported dry unit weight for each normal stress condition. A fresh geomembrane specimen was trimmed from the bulk sample of geomembrane provided by the manufacturer and attached to the lower shear box with mechanical compression clamps. Fresh specimens of the site clay were moisture-conditioned and compacted away from the geomembrane specimen by hand tamping to the reported dry unit weight for each normal stress condition and then placed on the geomembrane for testing. This set-up caused shearing to occur at the site clay-geomembrane interface. Each test specimen was tested under as-placed moisture conditions.

The target dry unit weight and moisture content conditions for the site clay and the drainage sand used in Test Series 2 were specified by OHM based on the project specifications. For the site clay, the target dry unit weight and moisture content corresponded to 95 percent of the maximum dry unit weight and 3 percentage points wet of the optimum moisture content, based on the results of standard Proctor compaction testing provided by OHM. For the drainage sand, the target dry unit weight and moisture content corresponded to 95 percent of the maximum dry unit weight and the optimum moisture content, based on the results of standard Proctor compaction testing provided by OHM. The reported values of dry unit weight for each soil specimen were determined by measuring the as-placed volume of the soil and dividing this volume into the calculated total dry weight of the soil specimen.

Other features of the testing procedure included the following:

- a freshly remolded 3-in. (75-mm) thick layer of concrete sand was used as a bedding layer below each test interface in Test Series 1; the concrete sand was compacted by hand tamping to a relatively dense state under dry conditions;

- each test specimen was sheared at a constant displacement rate immediately after application of the normal stress used for shearing;

- the direction of shear for each interface direct shear test was in the direction of manufacture (machine direction) of the geosynthetic samples;

- each test was performed using a constant effective sample area (i.e., the plan area of the geosynthetic specimens were larger than that of the upper shear box), therefore, no area correction was required when computing normal and shear stresses; and

- each test was sheared until a constant, residual shear load was recorded.

TEST RESULTS

Soil Property Tests

The results of single-point standard Proctor compaction test on the site clay are summarized in Table 3. The results of pinhole dispersion test on the site clay are summarized in Table 4. Table 4 also includes the specific test conditions for the pinhole dispersion test.

Hydraulic Conductivity Test

The results of the rigid wall constant head hydraulic conductivity test on the drainage sand are summarized in Table 5. The table also includes the specific test conditions for the hydraulic conductivity test.

Interface Direct Shear Tests

The total-stress interface shearing resistance was evaluated for each applied normal stress. The test data were plotted on a graph of shear force versus horizontal displacement. The resulting plots are presented in Attachment 1 to this report. The peak value of shear force was used to calculate the peak shear strength. For this report, the residual shear strength was assumed to be equal to the stabilized, post-peak shear force measured at the end of each test.

The total-stress peak and residual shear strengths derived from the plotted test results are summarized in Table 6. These strengths were plotted on a graph of shear stress versus normal stress and the results were used to evaluate the total-stress peak or residual strength envelopes. A best-fit straight line was drawn through the three data points from each test series to obtain total-stress peak and residual friction angles and adhesions. The coefficient of correlation (R^2), a standard statistical indicator of how well the best-fit line matches the test data, was obtained for each best-fit line. The plots of the shear stress versus normal stress for each test series are also presented in Attachment 1. The friction angles, adhesions, and R^2 values derived from the plotted test results are summarized in Table 7.

For each test series, it is noted that the reported adhesion is the shear stress axis intercept of the best-fit straight line drawn through the test data on a plot of shear stress versus normal stress. This value may not be the true adhesion of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test series.

CLOSURE

The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions. The test results should not be used in engineering analyses unless the test conditions model the anticipated field conditions. The testing was performed in accordance with general engineering testing standards and requirements. This testing report is submitted for the exclusive use of PROTECO and OHM.

Mr. Rene Rodriguez
28 September 1994
Page 9

GeoSyntec appreciates the opportunity to conduct laboratory testing for PROTECO. If you have any questions about this report, or if you require additional information, please do not hesitate to contact any of the undersigned.

Sincerely,

Zehong Yuan, Ph.D.
Assistant Department Manager
Soil-Geosynthetic Interaction Testing

Robert H. Swan, Jr.
Department Manager
Soil-Geosynthetic Interaction Testing

Gary R. Schmertmann, Ph.D., P.E. (Georgia)
Project Engineer

Attachments

Copy to: Mr. Joseph A. Carris, OHM Remediation Services Corporation

TABLE 1

SUMMARY OF GENERAL TESTING CONDITIONS
INTERFACE DIRECT SHEAR TESTING
PROTECO
PROTECO HAZARDOUS WASTE UNITS

Test Number	Interface Tested(1)	Target Dry Unit Weight and Moisture Content of Site Clay(2)	Consolidation Stress (psf)	Time for Consolidation (hours)	Normal Stress During Shearing (psf)	Rate of Shear (in./min)
1	NSC PN3000SCN Geonet/20-mil Staff Smooth PVC Geomembrane Under Wet Conditions	N/A	400, 600, and 800	0	400, 600, and 800	0.2
2	Site Clay (P-2)/20-mil Staff Smooth PVC Geomembrane Under As-Placed Moisture Conditions	89.7 pcf and 28.8%	400, 600, and 800	0	400, 600, and 800	0.04

NOTES: (1) For Test Series 1, the geonet and upper surface of the geomembrane were wetted by pouring tap water on top of the geonet specimen prior to being sheared. For Test Series 2, the site clay was compacted away from the geomembrane specimen and then placed on the geomembrane for testing; each test specimen was tested under as-placed moisture conditions.

(2) N/A refers to test conditions which are not applicable to the test.

TABLE 2

SUMMARY OF INTERFACE DIRECT SHEAR TEST EQUIPMENT AND CONDITIONS PROTECO PROTECO HAZARDOUS WASTE UNITS

Test Series Number	Shear Box Size	TEST CONDITIONS[1]								Normal Stress During Shearing (psf)	Displacement Rate (in./min)
		γ_{di} (pcf)		ω_{ci} (%)		Consolidation Stress (psf)	Time for Consolidation (Hours)	ω_{cf} (%)			
		Clay	Sand	Clay	Sand			Clay	Sand		
1	12" x 12"	N/A	N/A	N/A	N/A	400 600 800	0 0 0	N/A	N/A	400 600 800	0.2 0.2 0.2
2	12" x 12"	89.7 89.3 89.8	114.7 114.5 114.9	28.9 29.2 29.2	10.9 10.9 10.9	400 600 800	0 0 0	28.7 29.1 29.0	10.7 10.7 10.7	400 600 800	0.04 0.04 0.04

NOTE: (1) γ_{di} refers to initial dry unit weight of soil specimen.
ω_{ci} refers to initial moisture content of soil specimen.
ω_{cf} refers to final moisture content of soil specimen.
N/A refers to test data which is not applicable to the test.

TABLE 3

SOIL COMPACTION TEST RESULTS
PROTECO
PROTECO HAZARDOUS WASTE UNITS

Soil Sample Tested	Compaction Characteristics[1] ASTM D 698	
	Dry Unit Weight (pcf)	Moisture Content (%)
Site Clay	94.0	25.1

NOTE: (1) The single-point standard Proctor compaction test was conducted to confirm the compaction test results of the site clay (OHM sample number P-2) provided by PROTECO.

TABLE 4

SUMMARY OF THE PINHOLE DISPERSION TEST RESULTS
PROTECO
PROTECO HAZARDOUS WASTE UNITS

Soil Sample Tested	Test Specimen Initial Conditions		Pinhole Dispersion ASTM D 4647				
	Dry Unit Weight (pcf)	Moisture Content (%)	Maximum Hydraulic Head (in.)	Test Duration (min)	Dispersive Classification	Method	Remarks
Site Clay	90.7	29.5	15	25	ND1 (Perfectly Clear)	A	No Visible Erosion

TABLE 5

SUMMARY OF HYDRAULIC CONDUCTIVITY TEST RESULTS
PROTECO
PROTECO HAZARDOUS WASTE UNITS

Test Number	Test Specimen	Fluid Type	Initial Specimen Dimension		Initial(1) Specimen Condition		Hydration/(2) Saturation Stress (psi)	Effective(2) Consolidation Stress (psi)	Hydraulic Gradient (i)	Final Moisture Content (%)	Hydraulic Conductivity (cm/sec)
			Diameter (in.)	Height (in.)	γ_{di} (pcf)	ω_{ci} (%)					
1	Drainage Sand	Tap Water	6.0	6.0	114.7	10.8	N/A	N/A	0.2	14.2	2.2×10^{-4}

Notes: (1) γ_{di} refers to initial dry unit weight of the drainage sand material specimen.
ω_{ci} refers to initial moisture content of the drainage sand material specimen.

(2) N/A refers to data which is not applicable to test.

TABLE 6

INTERFACE DIRECT SHEAR TEST RESULTS
MEASURED PEAK AND RESIDUAL TOTAL SHEAR STRENGTHS
PROTECO
PROTECO HAZARDOUS WASTE UNITS

Test Series Number	Normal[(1)] Stress (psf)	Measured Peak Shear Strength (psf)	Measured Residual Shear Strength (psf)	Reference Attachment Figure Number
1	400 600 800	143 183 230	117 151 204	1-1 and 1-2
2	400 600 800	197 253 297	123 142 160	1-3 and 1-4

NOTE: (1) Test specimens were sheared immediately after application of normal stress.

TABLE 7

INTERFACE DIRECT SHEAR TEST RESULTS
MEASURED TOTAL STRESS SHEAR STRENGTH PARAMETERS
PROTECO
PROTECO HAZARDOUS WASTE UNITS

Test Number	Interface Tested[1]	Normal[2] Stress (psf)	Peak Strength[3]			Residual Strength[3]		
			Friction Angle	Adhesion (psf)	R^2	Friction Angle	Adhesion (psf)	R^2
1	NSC PN3000SCN Geonet/20-mil Staff Smooth PVC Geomembrane Under Wet Conditions	400 to 800	12°	55	0.998	12°	27	0.984
2	Site Clay (P-2)/20-mil Staff smooth PVC Geomembrane Under As-Placed Moisture Conditions	400 to 800	14°	99	0.995	5°	86	1.000

NOTES: (1) For Test Series 1, the geonet and upper surface of the geomembrane were wetted by pouring tap water on top of the geonet specimen prior to being sheared. For Test Series 2, the site clay was compacted away from the geomembrane specimen and then placed on the geomembrane for testing; each test specimen was tested under as-placed moisture conditions.

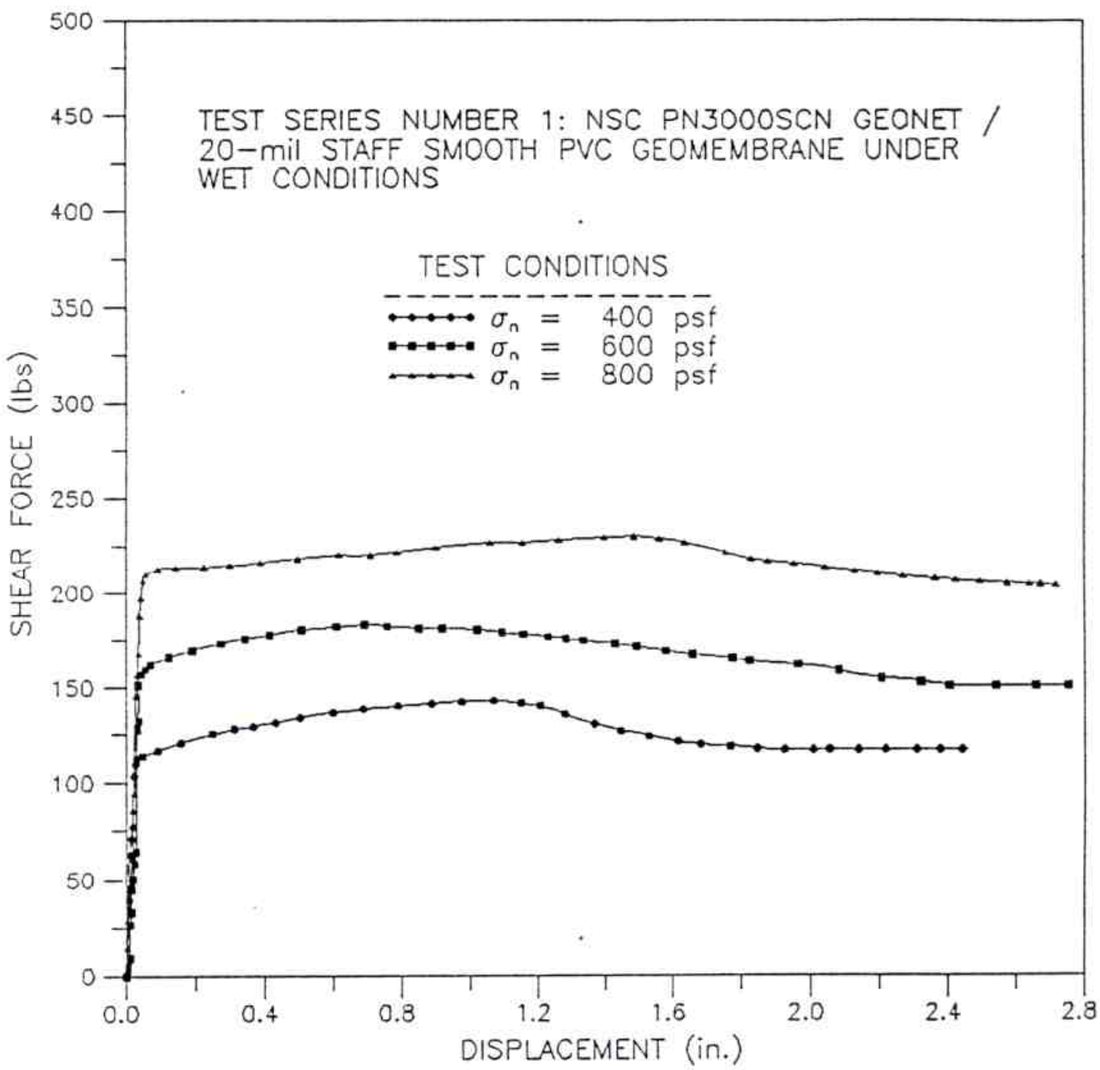
(2) Test specimens were sheared immediately after application of the normal stress used for shearing.

(3) The reported value of adhesion may not be the "true adhesion of the interface and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test. The value of R^2, the coefficient of correlation, provides an indication of how well the best-fit shear strength parameters match the test data.

ATTACHMENT 1

INTERFACE DIRECT SHEAR TEST DATA

PROTECO
INTERFACE DIRECT SHEAR TESTING



NOTE: The shear box size was 12 in. by 12 in.(300 mm by 300 mm), and the contact area remained constant throughout the entire test.

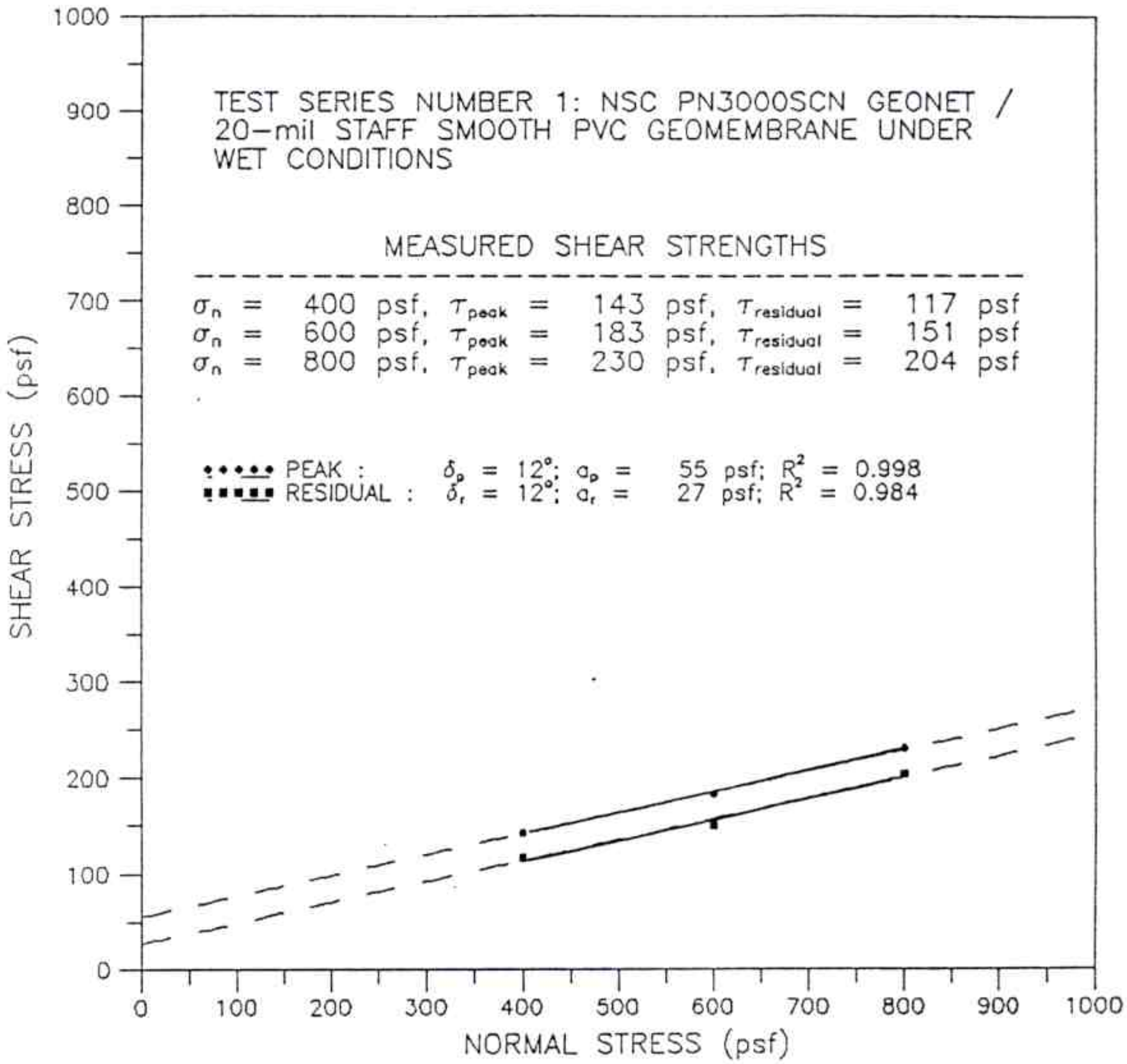
DATE TESTED: 2 AUGUST 1994

GEOSYNTEC CONSULTANTS

GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	1-1
PROJECT NO.	GL13612
DOCUMENT NO.	GEL94376
PAGE NO.	

PROTECO
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

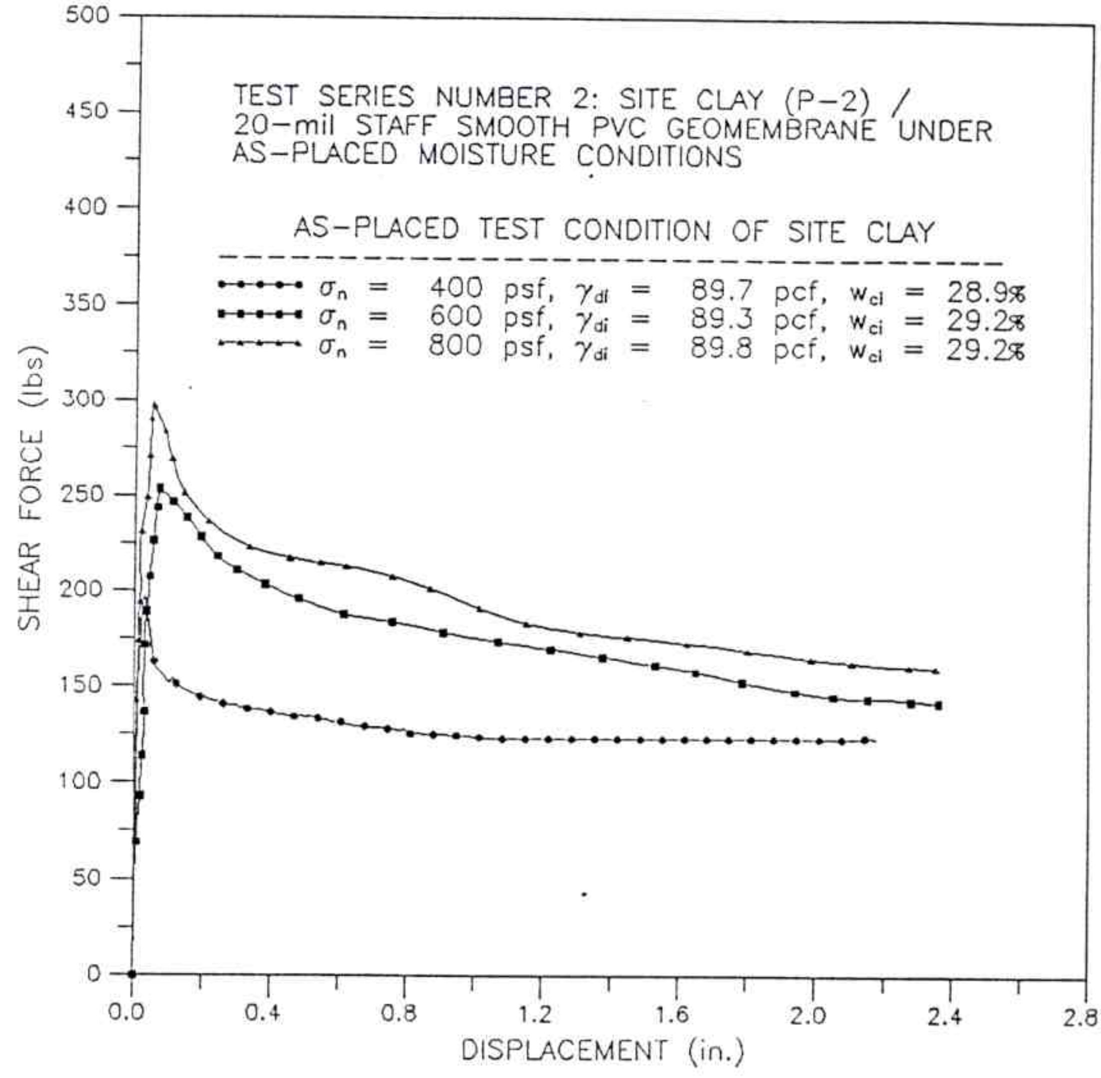
DATE TESTED: 2 AUGUST 1994

GEOSYNTEC CONSULTANTS

GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	1-2
PROJECT NO.	GL13612
DOCUMENT NO.	GEL94376
PAGE NO.	

PROTECO
INTERFACE DIRECT SHEAR TESTING



NOTE: The shear box size was 12 in. by 12 in.(300 mm by 300 mm), and the contact area remained constant throughout the entire test.

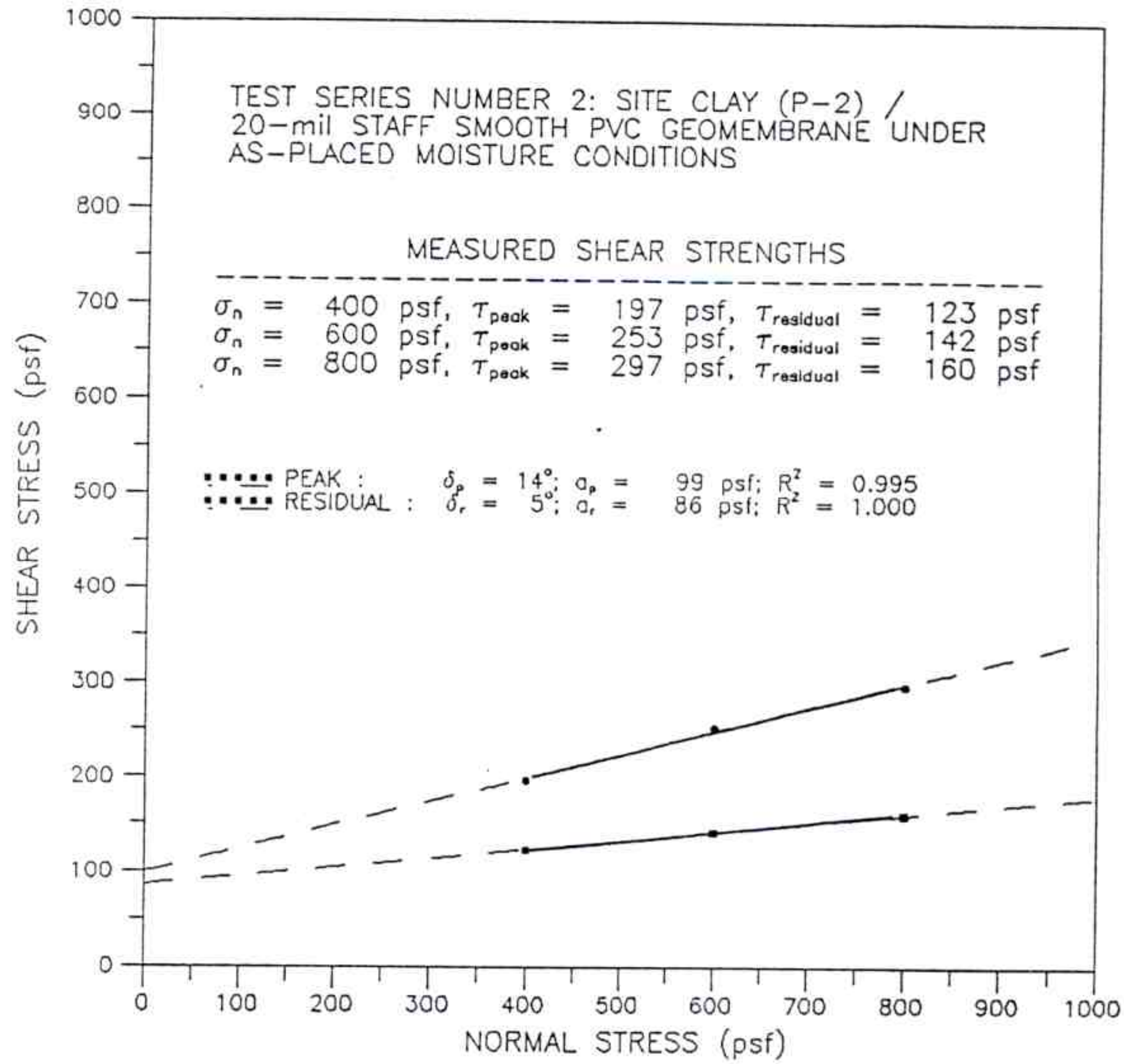
DATE TESTED: 10 AUGUST 1994

GeoSyntec Consultants

GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	1-3
PROJECT NO.	GLI3612
DOCUMENT NO.	GEL94376
PAGE NO.	

PROTECO
INTERFACE DIRECT SHEAR TESTING



NOTE: The reported value of adhesion may not be the true adhesion of the interface, and caution should be exercised in using this adhesion value for applications involving normal stresses outside the range of stresses covered by the test.

DATE TESTED: 10 AUGUST 1994

GEOSYNTEC CONSULTANTS
GEOMECHANICS AND ENVIRONMENTAL LABORATORY

FIGURE NO.	1-4
PROJECT NO.	GLI3612
DOCUMENT NO.	GEL94376
PAGE NO.	

APPENDIX F

LABORATORY TEST RESULTS PROTECO HAZARDOUS WASTE UNITS, PENUELAS, PUERTO RICO

CARIBBEAN SOIL TESTING CO. INC.

LABORATORY TEST RESULTS
PROTECO
HAZARDOUS WASTE UNITS
PEÑUELAS, PUERTO RICO

CARIBBEAN SOIL TESTING CO. INC.

SOIL AND MATERIALS TESTING LABORATORY

OFFICES

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TABLE OF CONTENTS

LABORATORY REPORT

TO : MR. RENE RODRIGUEZ
VICE-PRESIDENT
PROTECO
P.O. BOX 850
PEÑUELAS, PUERTO RICO 00624

SUBJECT : LABORATORY TEST RESULTS,
PROTECO HAZARDOUS WASTE UNITS,
PEÑUELAS, PUERTO RICO.

DATE : AUGUST 9, 1994

1.0 INTRODUCTION

This report presents the results of the laboratory tests performed in some of the soil samples secured from eight test holes (GT 1 through GT 8) and on 5 grab samples (P-1 through P-5) delivered to our laboratory and proceeding from the referenced project site at Peñuelas, Puerto Rico.

The work consisted in the performance of eight test holes for the securing of both, disturbed and undisturbed samples following the terms and conditions as contained in **our accepted proposal dated April 29, 1994.**

The test holes were drilled by the power auger method of drilling using a CME 45 drilling rig. Soil samples were taken at vertical intervals not exceeding five (5) feet by means of a 1-3/8" I.D. Split Spoon Sampler according to ASTM Designation D-1586-84 and/or D-1452-80 or through the Denison Sampler.

The analyses performed correspond to Unconfined Strength, Permeability, Liquid and Plastic Limits, Proctor Densities, Gradation and Consolidation Tests.

The amount of borings as well as the tests and the samples analyzed was established by others.

In addition, a general evaluation of the stability of some existing slopes was performed.

The following **TABLE I** contains information of the borings location and elevation.

TABLE I
SOIL TEST BORINGS
LOCATION AND ELEVATION

Boring Number	*Lambert Coordinates (ft)*		*Elevation (ft)*
	Northing	*Easting*	
GT-1	*66,009.50*	*407,017.50*	*316.922*
GT-2	*66,242.25*	*407,173.50*	*326.455*
GT-3	*66,024.50*	*407,146.90*	*350.264*
GT-4	*66,435.00*	*407,558.50*	*380.714*
GT-5	*66,710.00*	*407,091.00*	*301.244*
GT-6	*66,068.00*	*406,785.00*	*270.201*
GT-7	*68,001.30*	*406,920.20*	*390.125*
GT-8	*68,127.50*	*407,330.50*	*406.838*

FIGURE I is a copy of the site topographic map, with the borings location shown on it.

2.0 REPORT

2.1 General Site Location and Description

The site under study is located about 1.5 km, East of Tallaboa Poniente Ward, Peñuelas, Puerto Rico.

The landscape area where the existing facilities are located (refer to **FIGURE 2** which is a copy of the Peñuelas quadrangle where the site has been depicted on it) is mountainous with rounded hilltop and gravelly footslopes. This area is categorized as semiarid uplands with steep slopes, rapid runoff and erosion hazard.

At the site, slopes with height varying from about 40 to 50 feet and with inclinations varying from about 20° to 60° have been observed in which most of the most steeper correspond to artificial cuts. **PLATE A AND B** contain photos showing the site general conditions.

2.2 Geology Settings

Based on the **U.S.G.S. Geologic Map of the Peñuelas and Punta Cuchara Quadrangles of Puerto Rico**, as prepared by Richard D. Krushensky and Watson H. Monroe, the geology of the area has been described as the **Chalk Member (Tjc)**, the **Ponce Limestone (Miocene) (Tp) and Alluvium (Qa)** described as:

CHALK MEMBER - *"White to very pale orange clayey chalk and bedded chalky limestone; contains many small Foraminifera; ranges in thickness from about 50* *m in the western part of the quadrangle, where it overlies a reef limestone, to about 300 m near Ponce." Strikes of about 30° Northwest and dips some 10° have been measured on the bedding planes at this formation."*

PONCE LIMESTONE - *"Very pale orange to grayish-orange generally crystalline calcarenite. Contains abundant internal molds of fossils, specially mollisds and solitary corals; also contains shells of the echinoid Clypeaster cubensis, oysters and the tests of Foraminifera such as Marginopora sp. and Gypsina sp. Thickness is more than 200 m and may be as much as 850 m near the southwest corner of the Peñuelas quadrangle."*

ALLUVIUM - *"Cobbles, pebbles, sand, clay, and sandy clay. Thickness variable, but probably as much as 50 m in the area southwest of Ponce and in the southern part of the Rio Tallaboa."*

FIGURE 3 is a copy of the geologic map of the Peñuelas quadrangle with the site depicted on it.

From the Soil Survey of Ponce Area of Southern Puerto Rico prepared by the United State Department of Agriculture, Soil Conservation Service in cooperation with the University of Puerto Rico, College of Agricultural Sciences the soils of the area have been classified on the Map Sheet No. 28 as the Aguilita and Yauco series as follows.

Aguilita series

AgF- Aguilita gravelly clay loam, 20 to 60 percent slopes. *This steep to very steep soil is on side slopes and ridges in the semiarid area. It generally is in areas of more than 100 acres. It is the soil described as representative of the series. Cobbles and stones are scattered on 5 to 15 percent of the surface.*

Included with this soil in mapping are small areas of soils that do not have gravel in the surface layer and many small severely eroded spots where soft limestone is in the surface layer.

Runoff is rapid to very rapid, and erosion is a hazard.

AgD- Aguilita gravelly clay loam, 12 to 20 percent slopes. *This moderately to steep soil is on side slopes, foot slopes, and rounded hilltops of the semiarid uplands. This soil is similar to the one described as representative of the series, except surface layer is slightly thicker. Cobbles and stones are in small areas have 10 to 20 percent of the surface. Also included are not gravelly in the surface layer.*

Included with this soil in mapping are small areas of soils that do not have gravel in the surface layer.

Runoff is rapid to very rapid, and erosion is a hazard.

Yauco series

YcC- Yauco silty clay loam, 5 to 12 percent slopes.

This is a strongly sloping soil on small rounded hills and foot slopes below the limestone hills. Runoff is medium and erosion is a hazard.

These soils formed in transported in moderately fine textured sediment that was derived from limestone.

2.3 General Slope Stability

It has been also requested from us to submit general recommendations in relation to the stability of the cut slopes prevailing at the site.

For stability evaluations, Test Holes GT-3 and GT-4 were performed to obtain some engineering properties of the soils. The results of such test holes and the corresponding tests performed on some of the secured soil samples, it has been encountered that the uppermost soils consist of hard to very hard, light gray to grayish-white chalky clayey silt and should correspond to the Chalk Member Limestone.

The N value of the Standard Penetration Test vary from about 33 to more than 100 blows per foot.

The shear resistance of any ideal plastic materials is defined by two main components; the cohesion (c) and the angle of shearing resistance (ϕ).

An Unconfined Compressive Strength test ($\phi=0$) performed on sample from a depth of 3' to 5' of Test Hole No. GT-4, revealed an cohesion strength of about 400 psf. Similar tests on samples from other test holes revealed cohesion strength as high as 1,500 psf.

Since no Triaxial Tests were performed on the secured samples, from available literature, the soil strength parameters were estimated. For normally loaded CH to CL soils with a PI of about 35 to 40, the effective angle of internal friction (ϕ) can be estimated to vary from about 20° to 30°.

Slope stability analyses by both the limit equilibrium and limit analysis methods show that considering a ϕ angle of 25° and a soil unit weight of 100 pcf were performed to determine the minimum cohesion value necessary to have slopes with a factor of safety of 1.20.

Such analyses revealed the following minimum cohesion values for different slopes heights.

_T A B L E I I_
REQUIRED COHESION FOR STABLE SLOPES
WITH INCLINATION OF 60°

Slope Height in Feet	*Required Cohesion in PSF*
20	*200*
30	*300*
40	*400*
60	*600*

As previously mentioned, at the area under consideration (Test Holes GT-3 and GT-4), stable cut slopes as high as 30' and with inclinations as steep as about 60° have been stable for long time. From this condition it can be concluded that a minimum cohesion value of 300 psf should prevail in this material.

Therefore, existing cut slopes 30 feet or less in height, with inclinations as steep as 60° and with soil conditions similar to those encountered at Test Holes GT-3 and GT-4, should be stable. Nevertheless, it is well known that many factors may contribute in reducing the strength of the soil material, in which the most important is saturation of the soil that directly affect the effective strength and cohesion of the soil mass.

For partially saturated soils, the present condition of the slopes at the site, an apparent cohesion results from capillary forces which provide a temporary strength which is lost upon either saturation or by excessive drying of the soils.

Therefore, proper measures shall be taken to reduce the infiltration of water, erosion or excessive drying of the slopes. If these measurements cannot be implemented, then some small landslides, specially during periods of heavy rains cannot be discarded. In order to reduce to a minimum the possibility of partial landslides, then cut slopes shall not be made steeper than 27° (2H to IV).

In order to confirm the assumed soil parameters, it would be recommendable the performance of several triaxial tests to determine in a more precise manner, the strength parameters of the soils.

It should be further mentioned that different subsoil conditions might prevails in cut slopes occurring within the alluvium soils. Therefore, proper identification of the different geologic formations of the area should be made.

2.4 Slope Stability of Natural Slopes

2.4.1 Inherent Stability of Slopes

Slopes may be divided into two broad categories, **"natural"** - i.e., occurring solely through the forces of nature - and manmade. Natural slopes are formed by , and exist due to, geologic processes such as those which cause the uplifting or subsidence of areas, including the action of wind, water ice, and particularly gravity. With the surface of the earth constantly in a state of change due to natural processes, these latter forces may also create new slopes, e.g., through the cutting of channels by flowing water. In evaluating the stability of slopes, the force due to the gravity must be given prime consideration; whenever the forces which tend to eliminate a slope are not effectively resisted, a landslide occurs. Depending on circumstances, a landslide may be sudden or gradual, and it may affect an extensive or limited mass of the slope. In all instances, the result in a movement of material from a higher to a lower elevation. Thus, once a slope appears, the stage is set for change, i.e., the continuous process of elimination of old slopes and formation of new ones. This is brought about by one or more forces acting on the slopes, always in the presence of gravity, which itself ten to flatten all slopes.

As previously mentioned, other influences besides gravity -- e.g., the effect of wind, or water may caused loads on slope surface; seepage of underground water; vegetation, chemical or temperature changes, occurrence of earthquakes and other environmental condition -- from, time to time impose upon the mass of slopes, forces of varying intensity, direction and duration. Most often these forces add to gravity forces by having a direction favoring a landslide; occasionally, however, the direction tends to maintain the stability of the slope. At all times, the composite effect of the forces acting on the slope (steady gravity force as well as the numerous forces of varying direction and intensity) is represented by a resultant driving force tending to cause a landslide. The slopes, in accordance with its geometry and the properties of component materials, provide resistance to the driving force. As long as this resistance is capable of balancing the driving forces, a landslide is prevented; when the resistance is overbalanced, the slope deteriorates until a new equilibrium is attained between resistance and driving force.

In summary, there are, at all times, driving forces with tend to cause a landslide, and resisting internal slope forces which tend to prevent a slide. When the driving forces increase and/or the maximum resisting forces decrease such that their ratio approaches unity, a slide becomes imminent.

The destructive driving forces may increase or decrease with time due to surface erosion, deposition, creep, minor slides, moisture infiltration, and manmade construction. The maximum resisting forces which can be develop may change due to erosion, manmade construction, the development of cracks or fissures, water pressures, and the changes in shear strength resulting from changes in water content and chemical action. The net effect is a change in the factor of safety of the slope (the maximum possible resistible force divided by the driving forces). Whenever the change reduces the factor of safety below unity, a landslide will follow, creating new, flatter slopes; whenever the change is an increase in the safety factor, the stability of the slope will improve.

While most landslides can be carefully analyzed after occurrence and a specific **"cause"** can be determined after the fact, actually a landslide developing during the history of the slope; generally the particular **"cause"** to which failure is attributed is in fact only the most prominent or final contributing factor which converts a seemingly stable slope into the site of a landslide. The number of variables is too great, and the interrelation too complex, for development at present of a rigorous basis for theoretical analysis of the stability of slopes and for prediction of impending landslides. On the other hand, it is possible to devise guidelines of substantial value when used in conjunction with an empirical analysis and seasoned with experience and sound understanding of the mechanics of landslides.

For the above expressed reason, it should be clearly stated, although natural slopes seem to be stable due mainly to apparent good resistance of the in-situ soil to resist driving forces, studies, as previously mentioned, have not been performed and prediction of impending mass movements cannot be made, thus neither can we guarantee their stability.

2.5 Laboratory Test Results

The results of the tests performed in our laboratory are contained in the following table.

TABLE III
TESTS RESULTS

Sample No.	wl (%)	PI (%)	Qu ksf	Standard Proctor		Permeability cm/sec^2	Gradation Percent passing Sieve No.		
				mc (%)	mdd (pcf)		10	40	200
P-1	47.0	31	2.7	26.1	93.8	3.7×10^{-8}	-	-	-
P-2	44.5	28	3.0	25.8	94.4	1.17×10^{-7}	-	-	-
P-3	65.5	38.5	6.6	31.0	85.8	4.6×10^{-8}	100	99	97*
P-4	66.5	41.5	--	25.2	89.4	3.2×10^{-8}	--	--	--
P-5	--	--	--	11.0	120.6	--	--	--	--
GT 1 8' - 10'	72.2	36.7	2.5	--	--	--	--	--	--
GT-2 28' - 31'	67.5	31.5	2.70	--	--	--	100	97	97*
GT-2 48' - 50'	59.5	33.0	3.00	--	--	--	--	--	--
GT-3 13' - 15'	75.5	48.0	--	--	--	--	100	100	97*
GT-4 3' - 5'	57.0	26.0	0.83	--	--	--	--	--	--
GT-5 8' - 10'	58.5	28.4	--	--	--	--	--	--	--
GT-6 3' - 5'	56.9	27.9	--	--	--	4.9×10^{-7}	--	--	--
GT-6 18' - 20'	41.2	24.2	--	--	--	--	--	--	--
GT-7 18' - 19'	38.5	22.5	--	--	--	--	100	91	68*

* *See also hydrometer test results.*

The results of the consolidation test performed on sample from 18 to 19 feet deep of boring GT-7 revealed the following soil parameters.

TABLE IV
CONSOLIDATION TEST RESULTS

Compression Index (Cc)	Swell Index (Cs)	Preconsolidation Pressure (Pc)	Initital Void Ratio (e_o)
0.15	0.033	5,100 psf	0.2994

Consolidation Tests on samples from Test Holes GT-5 and GT-6 were not performed, since it was impossible to prepare sample for testing.

Graphs of the Standard Proctor Tests, Unconfined Compressive Strengths, Gradation and Consolidation tests are included as an **Appendix** to this report.

Respectfully Submitted,

CARIBBEAN SOIL TESTING CO., INC.

ALFONSO VAZQUEZ CASTILLO,
President

AVC\mvc

Enclosure

Reference Number: 6373-94

APPENDIXES

APPENDIX I-1

The borings were made by the Hollow Stem Auger method. This drilling process consists of penetrating 2-1/4" I.D. x 5" O.D. hollow stem auger sections of seamless steel tube with a spiral flight, to which are attached a finger - type cutter head at the lower end and an adapter cap at the top. Through the center of the steel tube a drill stem is found. The stem is composed of drill rods attached with a center plug with a drag bit at the lower end. The adapter at the top of the drill stem and auger flight permit advancement of the auger with the plug in place. As the holes is advanced additional lengths of hollow stem flight and center stem are added as required. Soil samples are secured from the bottom of the hole, after removing the center stem and plug, by means of a 1-3/8" I.D. Split Spoon Sampler. While securing the soil samples, the Standard Penetration Test is performed and the "N" values obtained. This is the number of blows required to drive the sampling spoon a distance of one (1.0) foot into the ground with 140 lb hammer falling 30 inches. The "N" value gives an indication of the consistency of cohesive soils and the relative density of granular soils, as follows:

COHESIVE SOILS

"N" VALUES (blows/ft.)	CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (tsf)
less than 2	Very soft	Less than 0.25
2-4	soft	0.25 - 0.50
4-8	medium	0.50 - 1.00
8-15	stiff	1.00 - 2.00
15-30	very stiff	2.00 - 4.00
more than 30	hard	+ 4.00

APPENDIX I-2

GRANULAR SOILS

"N" VALUES (blows/ft.)	RELATIVE DENSITY
5 - 0	very loose
5 - 10	loose
10 - 30	medium
30 - 50	dense
over 50	very dense

The samples recovered with the split spoon sampler are known as disturbed samples, where the natural structure of the subsoil is broken in the sampling process. Thus, the soil particles recovered in the sampling device most frequently loosens the linking or cementing characteristics they possess in their natural position. For example, there are some relatively soft types of rock formations that can be sampled at least for some depths with a split spoon sampler. The material recovered in the spoon sampler is described in the boring log as fragments of the particular rock encountered. However, when open excavations are made it is found that the rock may be solid or massive and not fragmented.

Therefore, the description of the various strata contained in the test borings performed shall be used only as a guide for decisions regarding the rippability characteristics of the underlying materials.

APPENDIX I-3

ROTATORY DRILLING:

At that depth at which further penetration is not feasible by the jetting and chopping process, advancement of the hole is obtained by making use of the rotatory drilling method. This method is used to drill in consolidated or semi-consolidated materials. It consists as the name implies, of rotating a string of rods while continuous downward pressure is maintained through the rods on a tungsten carbide or diamond bit at the bottom of the hole. A number of different types of bits are used, most of which are capable of reducing stone or the most compact soil formations to small chips or particles. Water is forced down the rods to the bit and the return flow brings the cuttings to the surface. To drill into a rock a core barrel is attached between the bit and the string of rods. The drilled rock enters into the core barrel while the stream of water is circulated through the rods and barrel to the bits, thus serving as a coolant. At intervals of about 2 to 5 feet the barrel is brought to the surface, and the core is removed.

An estimate of the in-situ rock quality can be obtained from the correlation provided by the rock quality designation (RQD). The RQD is defined as the percentage ratio between the total length of pieces of core, 4 inch or longer, that are sound and hard and the length of core drilled on a given run.

The following table indicates the relationship of RQD and in-situ Rock Quality:

APPENDIX I-4

RQD (%)	ROCK QUALITY
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor

LABORATORY WORK:

Soil samples are classified according to their constituents and the following terminology is used to denote the percentage by weight of each component:

DESCRIPTION TERM	RANGE OF PROPORTION (%)
Trace	1 - 10
Some	10 - 20
Adjective (sandy, silty, clayey)	20 - 35
And	35 - 50

Granular soils are cohesionless soils consisting of boulders, gravel, sand, either separately or in combination.

Boulders are the constituents with average diameter larger than three (3) inches. Gravel ranges from fine (No. 10 sieve) to coarse (3 inches sieve). Sand particles are those passing No. 10 sieve and retained on No. 200 mesh. The silt particles range from

APPENDIX I-5

0.06 mm to 0.002 mm.

Cohesive soils are those soils which possess the characteristics of cohesion and plasticity. They may be granular soils as described above with the addition of clay or organic silt which cause cohesion and plasticity or may be clay or organic silt with no coarse components.

The clay fraction is comprised of clay minerals and in general has an average particle diameter of less than 0.002 mm.

The organic silt fraction is that portion with average particle diameter less than 0.06 mm. The clay and organic silt may occur separately or in conjunction. Both materials will exhibit plastic qualities within a certain range of moisture content, but the range will be greater in the case of clay.

Besides determining the constituents and color, each sample is carefully examined for stratifications, presence of secondary structures, shell, fibrous or disseminated peat, plasticity, etc.

Natural Moisture Content:

The natural moisture content is determined by finding the quantity of water present in the voids of the soil specimen in its natural condition and dividing it by the dry weight of the sample.

APPENDIX I-6

The weight of water is determined by subtracting the weight of a soil specimen in its natural condition from the weight of the specimen after being dried in an oven at 105^{0} for 24 hours.

Unconfined Compression Test:

The cohesive soil specimens obtained from split spoon samples cannot be considered as undisturbed samples, nevertheless, their unconfined compressive strength can be easily determined to obtain some information as to the shearing strength.

Unconfined compression tests were performed by subjecting cylinders of soil some three (3) inches high and 1.5 inches in diameter to axial deflections at a constant load and measuring the resistance stress developed in the soil.

The load on the sample was applied and measured by a scale and the deflection recorded on a strain dial calibrated in thousands of an inch.

Caribbean Soil Testing Co., Inc.
Soils and Materials Testing Laboratory
Tel. (809) 753-0147 / Fax. 759-7880

APPENDIX II-1

The information contained in this report regarding the location, type and/or other details describing the project is being presented as reported to ***Vázquez Castillo, Vázquez Agrait & Associates***, Geotechnical Engineers ***for Caribbean Soil Testing Co., Inc.*** The corresponding foundation recommendations are being based on these data and the subsoil information which provides test borings. The soil parameters and foundation recommendations contained in this report for the design of building substructures and earth-related structures of the project, in general, shall be used only by competent engineers in the field of Structural Design and Foundation Engineering, who shall incorporate an adequate safety factor in their design as a result of the inherent limitations of the test boring data. *Aware of these limitations, it is of paramount importance to report in writing to this office any change or modification introduced to the project after this report is completed to investigate the need to alter or modify our foundation recommendations. This office shall evaluate such changes and submit a written report confirming our original foundation recommendations or modifying them, if necessary. Failure to comply with these requirements will invalidate this report.*

The data presented in the enclosed logs and/or drawings apply at the borehole locations, and based on this, cross sections of the subsoil are prepared assuming that the samples include the worst and best soil conditions at the site. This may not be true. When profiles between boring logs are given, they shall be used for comparison purposes only, but they shall not be assumed to represent true intermediate conditions between borings.

APPENDIX II-2

After examining carefully the enclosed recommendations, the ***Owner and/or Petitioner of this report*** shall deliver a copy of it to the project designers for their information and guidance, as well as to the project Resident Engineer and to the contractors for the purpose of obtaining all possible evidence of unexpected variations in the subsoil, not necessarily encountered during the subsurface exploration. The Soil Consultant shall be informed immediately of any abnormal soil condition.

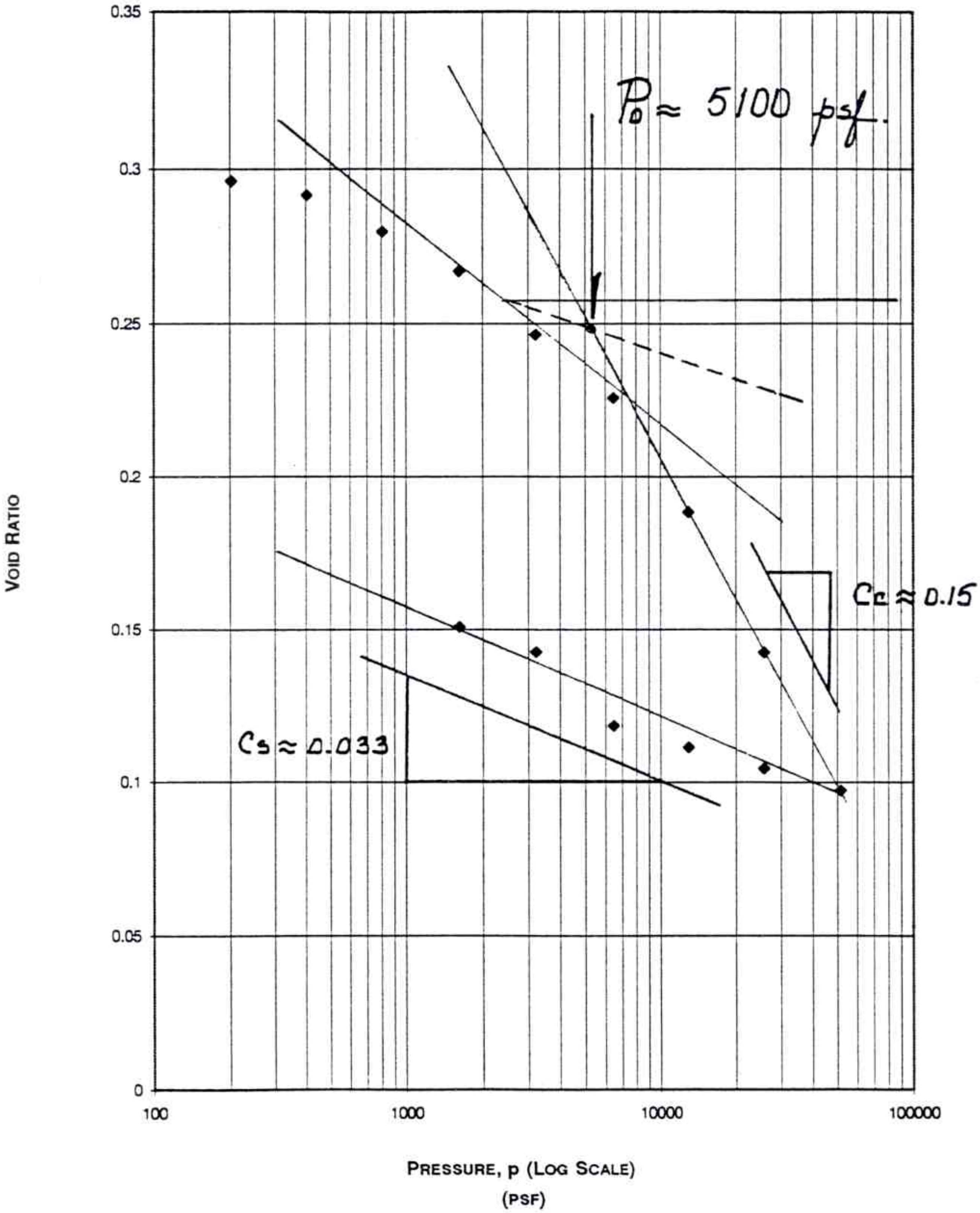
The position of the water surface on test borings shown on the enclosed logs indicate the phreatic level encountered either during the drilling process or shortly after the test boring is completed. The phreatic or underground water level is influenced by many factors and should not be assumed to have a permanent position. Foundation designs and works requiring to know and consider the fluctuations in the position of the water table shall include long range observations on deep wells. Where deep excavations are contemplated, as in the case of pumping stations, a study of artesian or sub-artesian aquifers should be made with deep test borings and pumping tests. Except otherwise stated in this report, the foundation works shall be made under the supervision of the Resident Engineer who shall be responsible for requiring compliance with the foundation recommendations of this report.

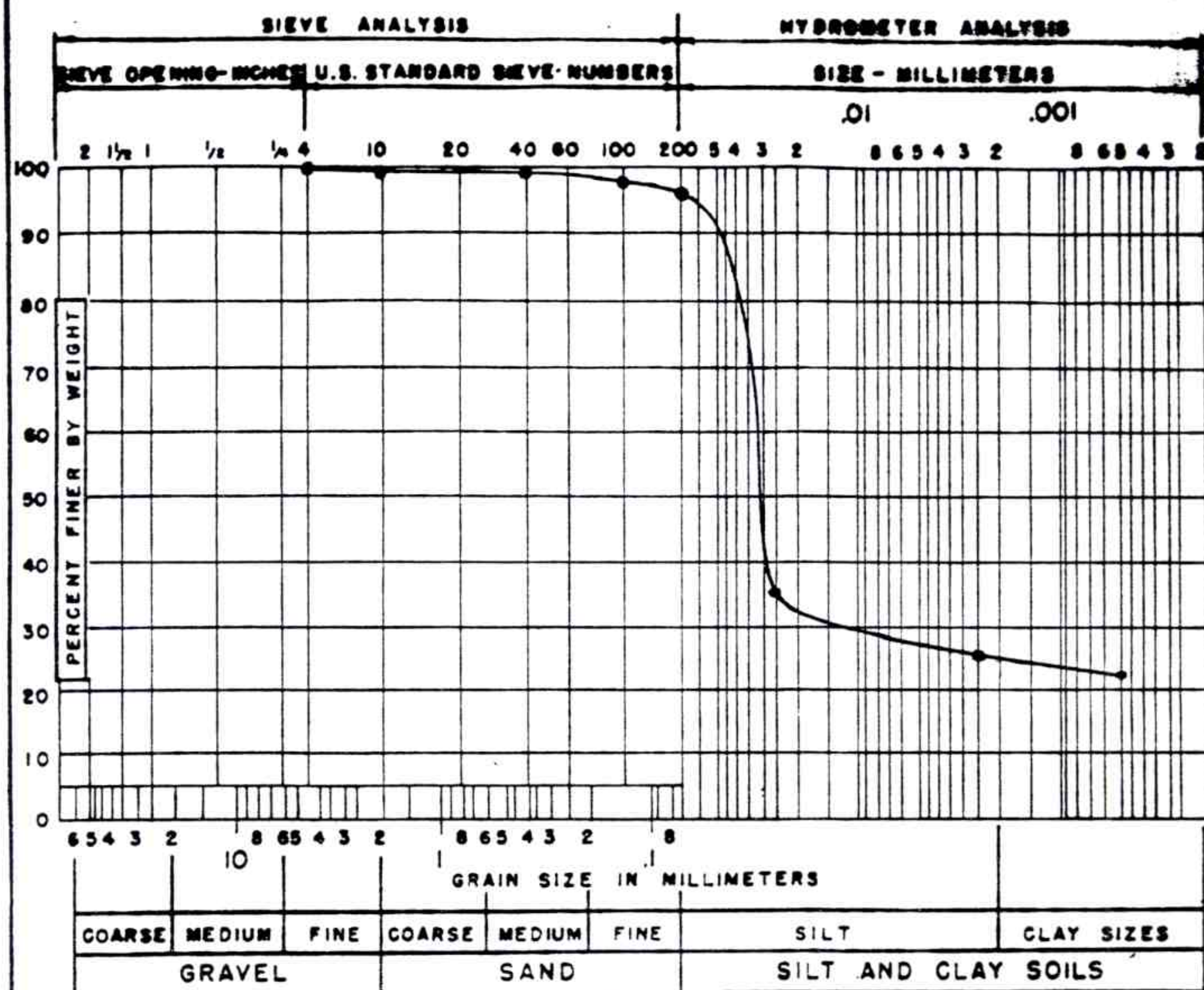
The Resident Engineer shall report to this Consultant any doubt or any abnormal soil condition which might be encountered during the construction or even after construction is completed to obtain solutions to such possible conditions.

Caribbean Soil Testing Co., Inc.
Soils and Materials Testing Laboratory
Tel. (809) 753-0147 / Fax. 759-7880

CARIBBEAN SOIL TESTING CO., INC.

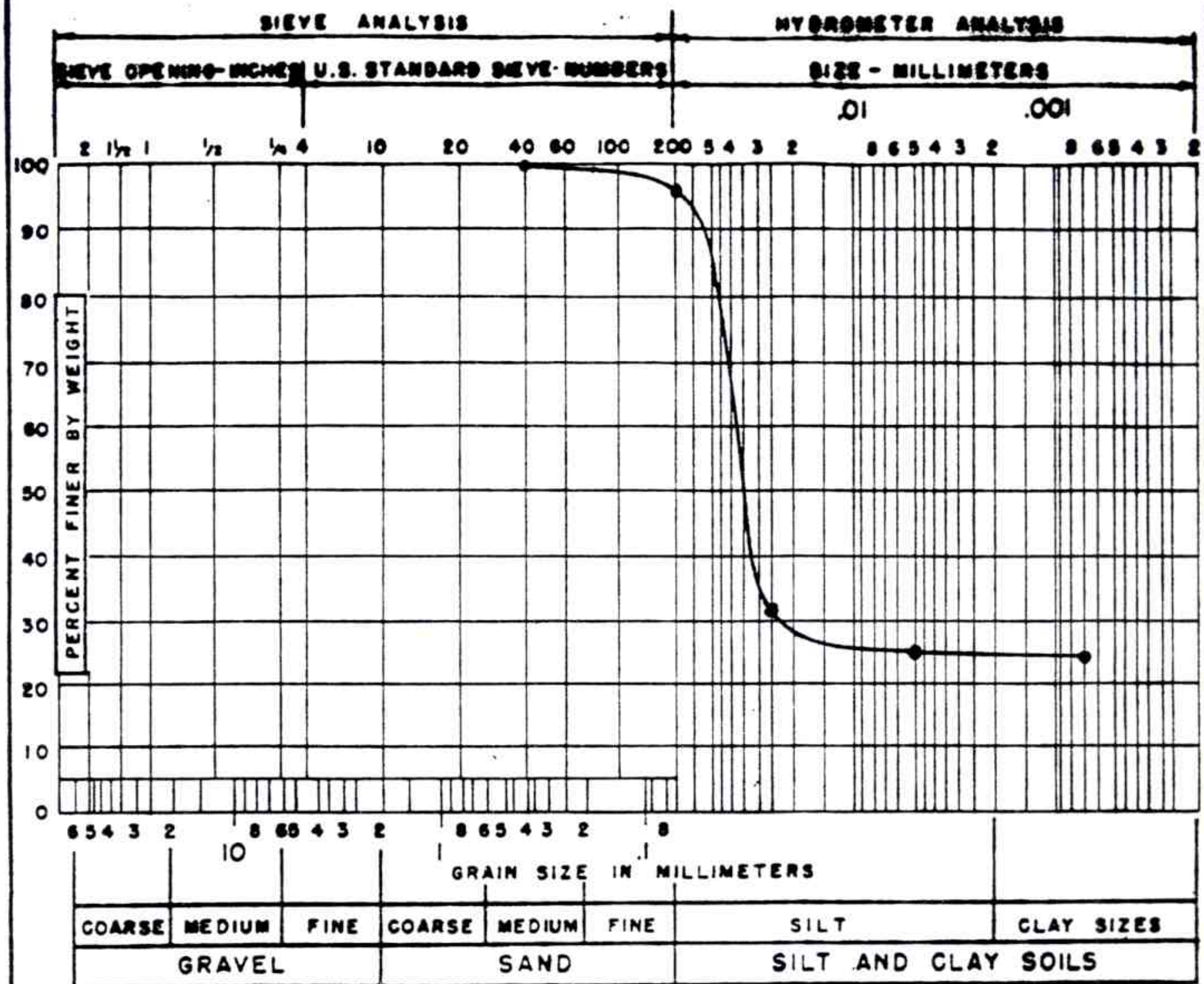
e-LOG p CURVE





CURVE NO.	SYM.	SAMPLE NUMBER	DEPTH	ELEV.	L.L.	P.I.	DESCRIPTION
1		GT-3	13'-15'		75.5	48.0	

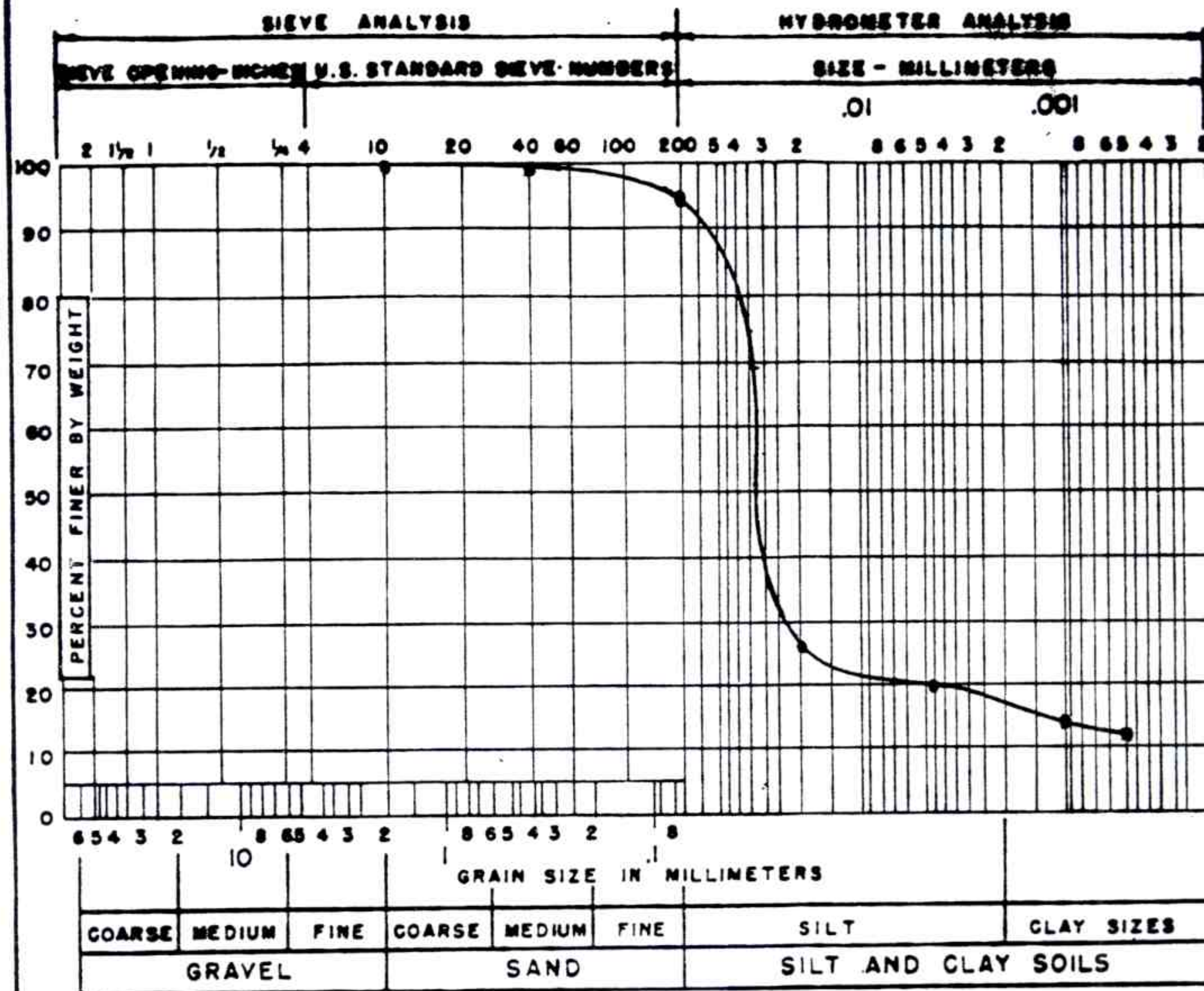
PROTECO		CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.
	GRAIN SIZE DISTRIBUTION	BY: / DATE: 8/02/94
		DWG. J.J.R.Q.



CURVE NO.	SYM.	SAMPLE NUMBER	DEPTH	ELEV.	L.L.	P.I.	DESCRIPTION
3		GT-2	28-31				

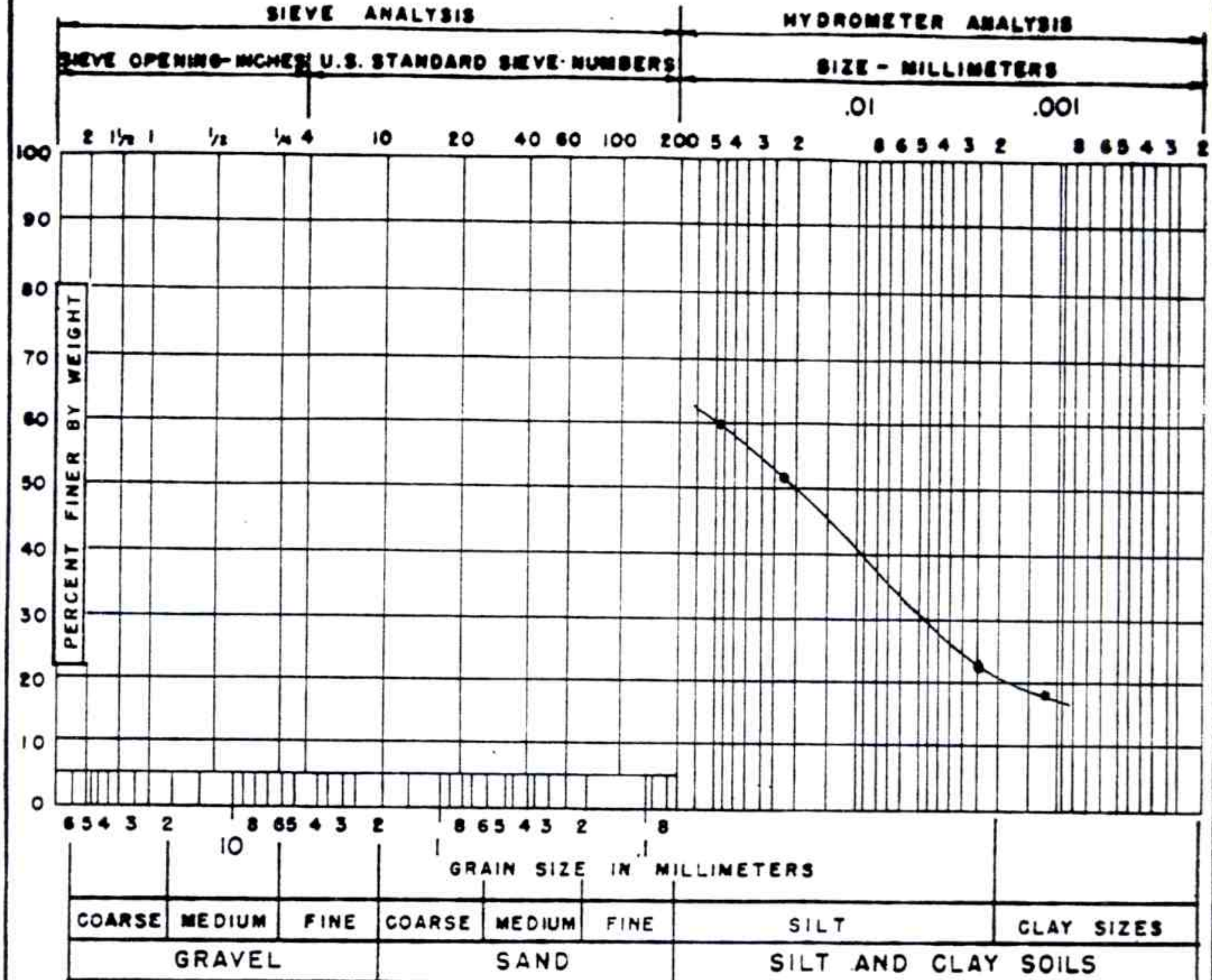
PROTECO	CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.

	GRAIN SIZE DISTRIBUTION	BY:	DATE: 8/02/94
		DWG. J.J.R.Q.	



CURVE NO.	SYM.	SAMPLE NUMBER	DEPTH	ELEV.	L.L.	P.I.	DESCRIPTION
2		BULK P-3	–				

PROTECO		CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.
	GRAIN SIZE DISTRIBUTION	BY: / DATE: 8/02/94 / DWG. J.J.R.Q.



CURVE NO.	SYM.	SAMPLE NUMBER	DEPTH	ELEV.	L.L.	P.I.	DESCRIPTION
		GT-7	18'-20'		38.5	22.5	CL

PROTECO

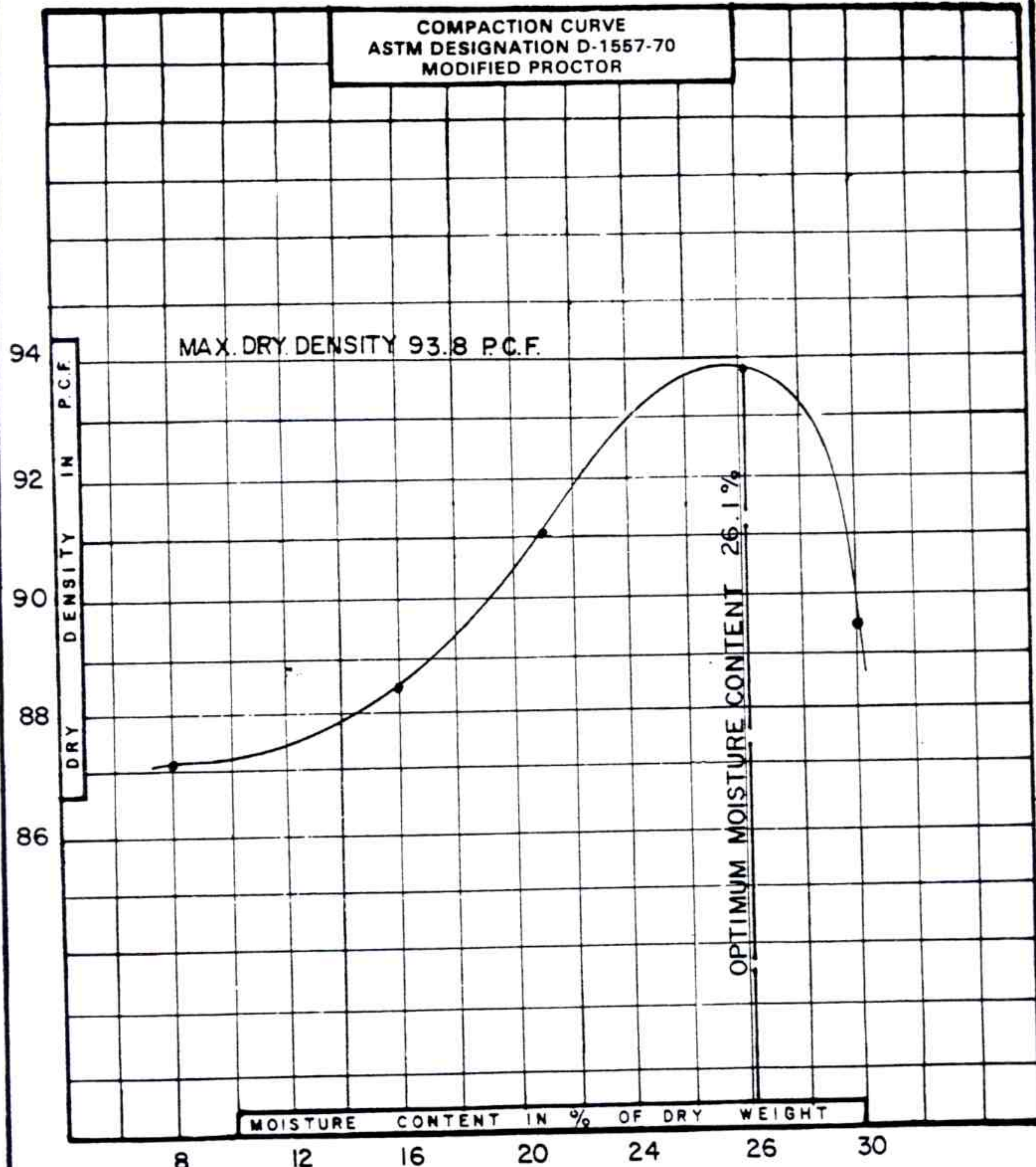
CARIBBEAN SOIL TESTING CO., INC.
Consulting Engineers, Hato Rey , P.R.

GRAIN SIZE DISTRIBUTION

BY:

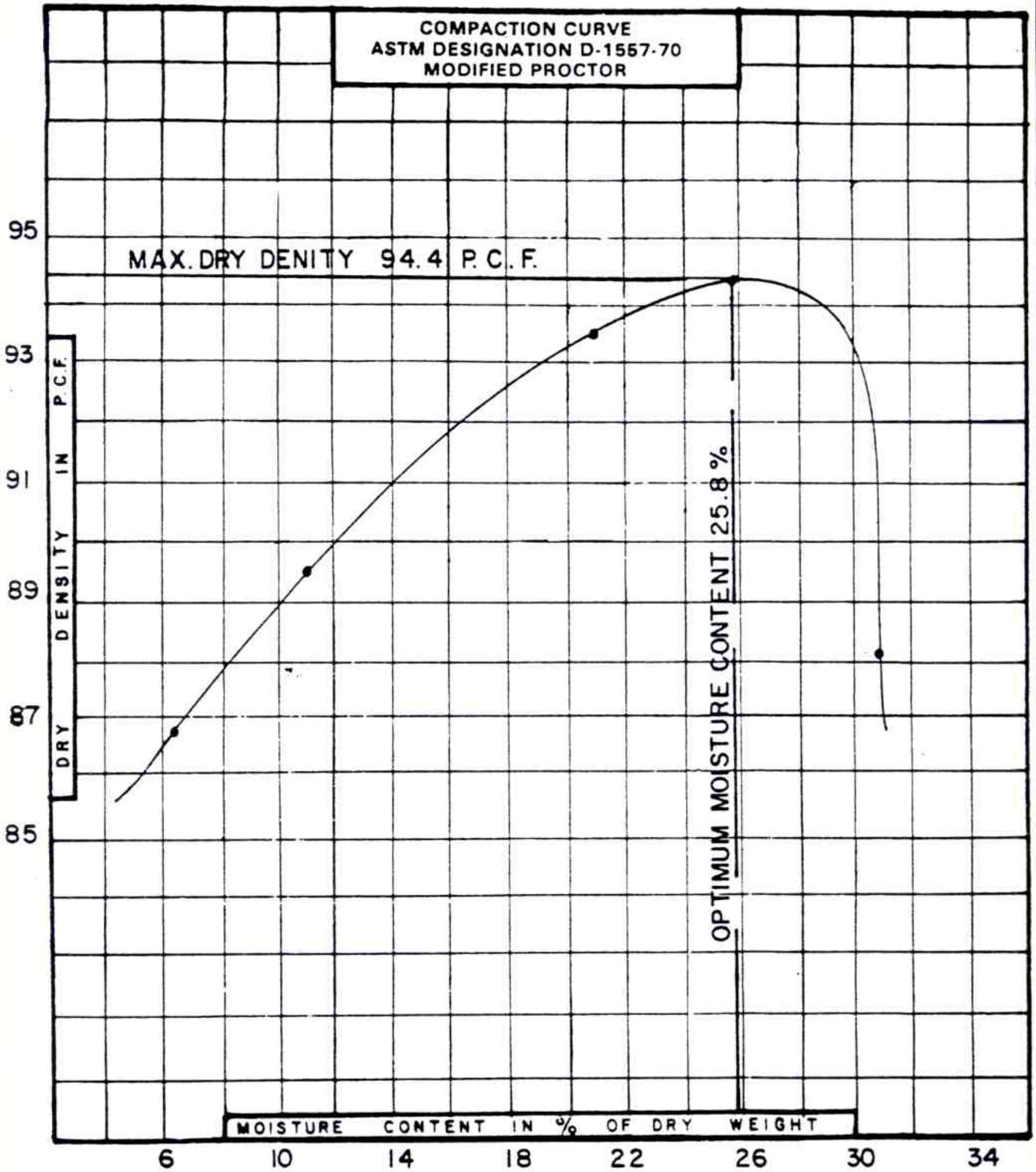
DATE: 8/05/94

DWG. J.J.R.Q.



CURVE NO.	SAMPLE DESCRIPTION	MAX. DRY DENSITY IN P.C.F.	OPTIMUM MOISTURE CONTENT IN %
P-1	LIGHT TAN TO LIGHT TONNISH BROWN SILTY CLAY, PLASTIC	93.8	26.1

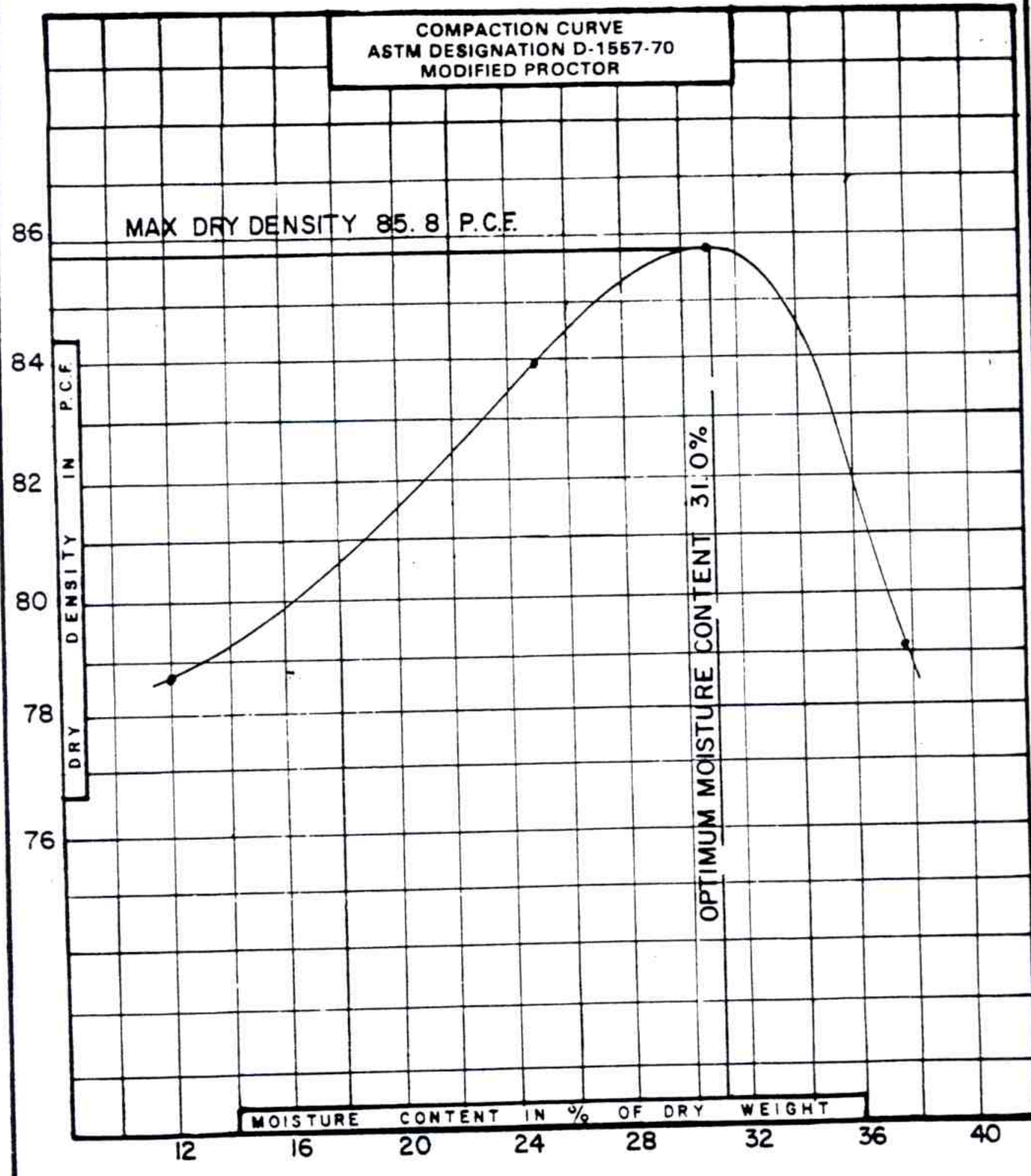
PROTECO		CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.
BY A. VAZQUEZ	DATE: 06/08/94	DWG. J. J.R.Q.



CURVE NO.	SAMPLE DESCRIPTION	MAX DRY DENSITY IN P.C.F.	OPTIMUM MOISTURE CONTENT IN %
P-2	LIGHT TAN SILT CLAY, PLASTIC	94.4	25.8

PROTECO	CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.

BY A.VAZQUEZ	DATE: 06/08/94	DWG. J.J.R.Q.

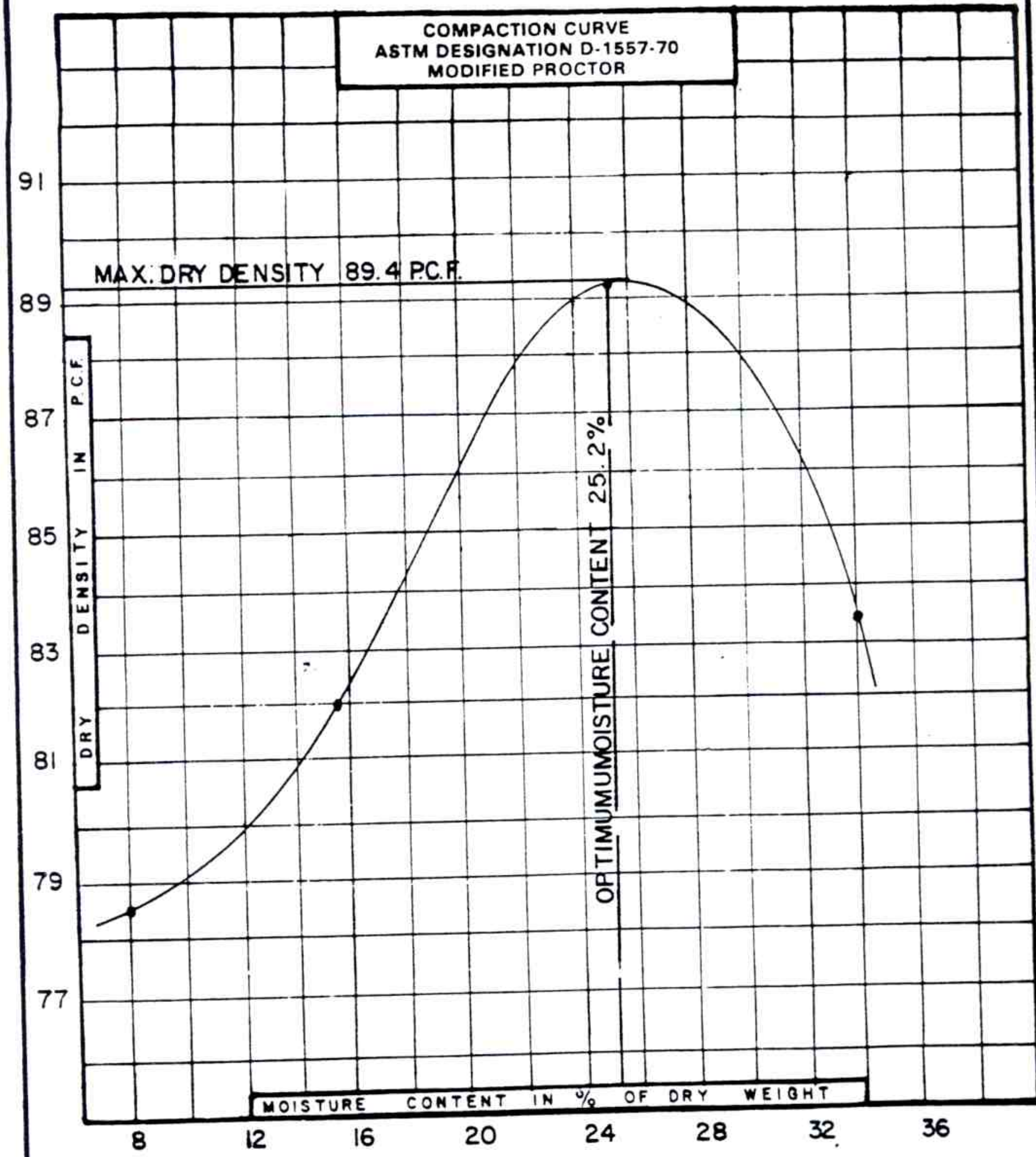


CURVE NO.	SAMPLE DESCRIPTION	MAX DRY DENSITY IN P.C.F.	OPTIMUM MOISTURE CONTENT IN %
P-3	LIGHT YELLOWISH TAN PLASTIC SILTY CLAY	85.8	31.0

LL 65.5,PI 38.5

PROTECO	CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.

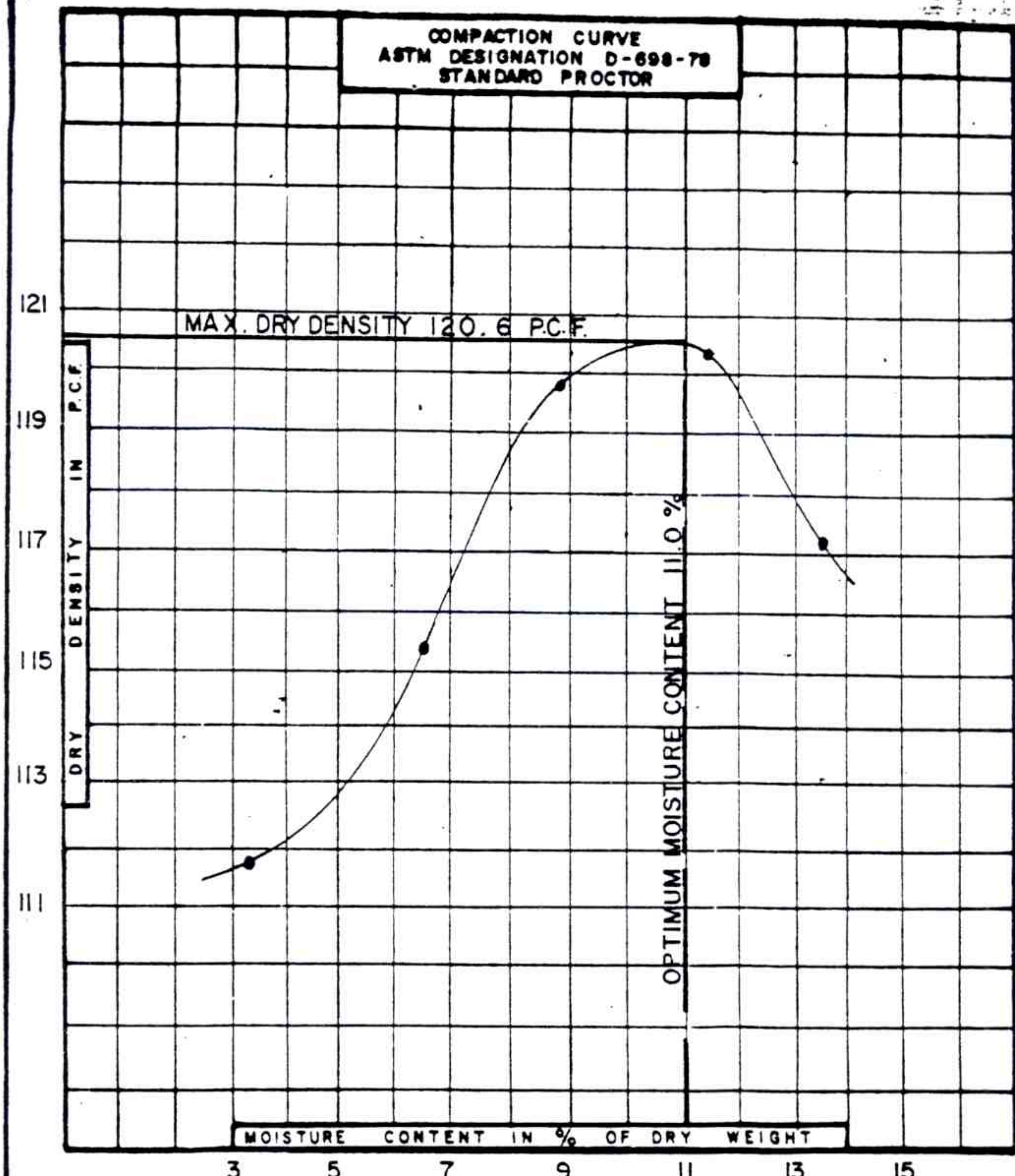
BY A.VAZQUEZ	DATE: 06/08/94	DWG. J.J.R.Q.



CURVE NO.	SAMPLE DESCRIPTION	MAX. DRY DENSITY IN P.C.F.	OPTIMUM MOISTURE CONTENT IN %
P-4	LIGHT TAN SILTY CLAY, PLASTIC	89.4	25.2

LL 66.5, PI 41.5

PROTECO		CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.
BY . A. VAZQUEZ	DATE: 06/08/94	DWG. J.J.R.Q.



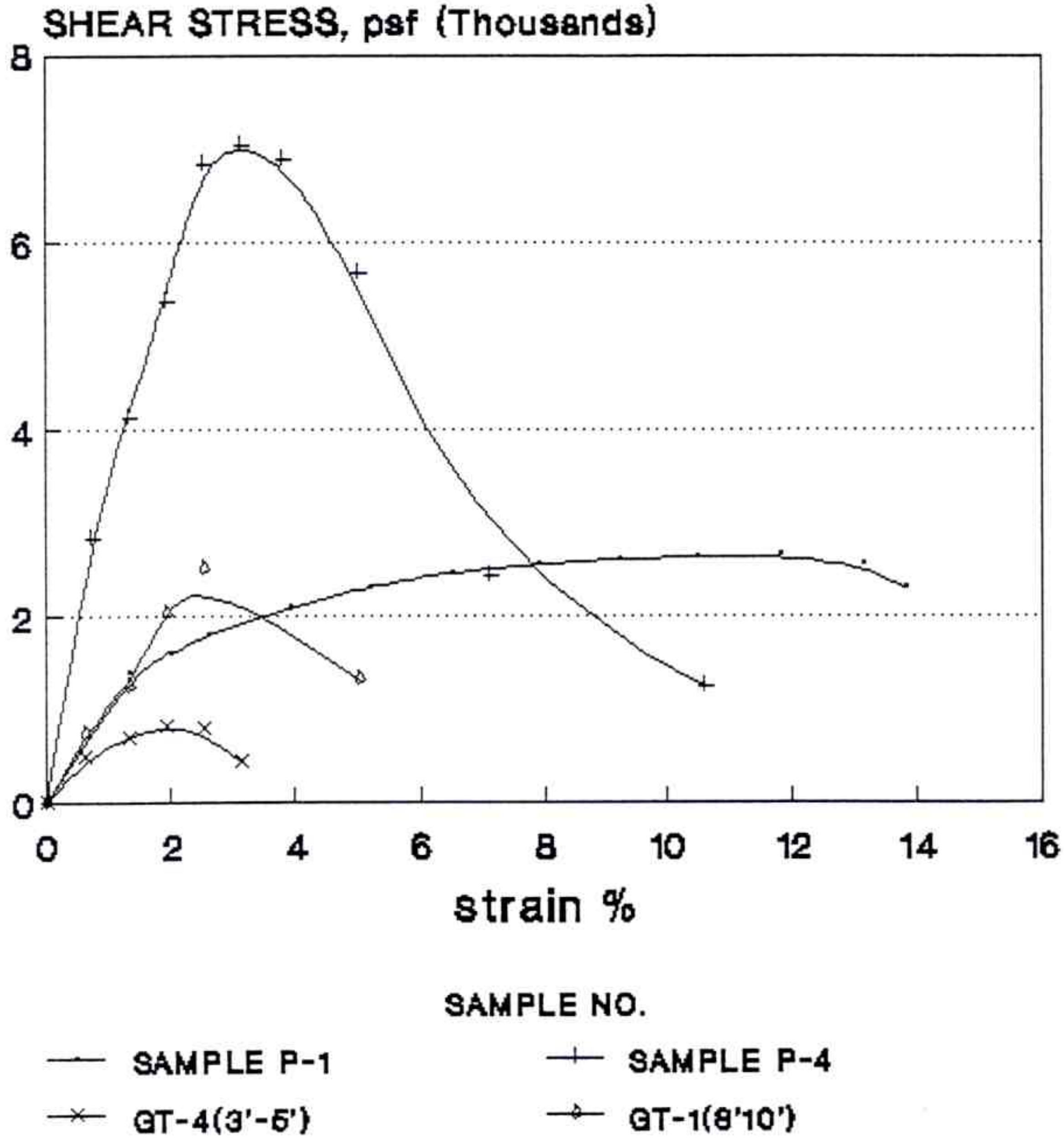
CURVE NO.	SAMPLE DESCRIPTION	MAX. DRY DENSITY IN P.C.F	OPTIMUM MOISTURE CONTENT IN %
P 5	DARK GRAYISH-BROWN, FINE TO MEDIUM SAND, SOME FINES	120.6	11.0

PROTECO	CARIBBEAN SOIL TESTING CO., INC. Consulting Engineers, Hato Rey, P.R.

BY. Á. RODRIGUEZ	DATE: 06/23/94	DWG. J.J. R.Q.

PROTECO

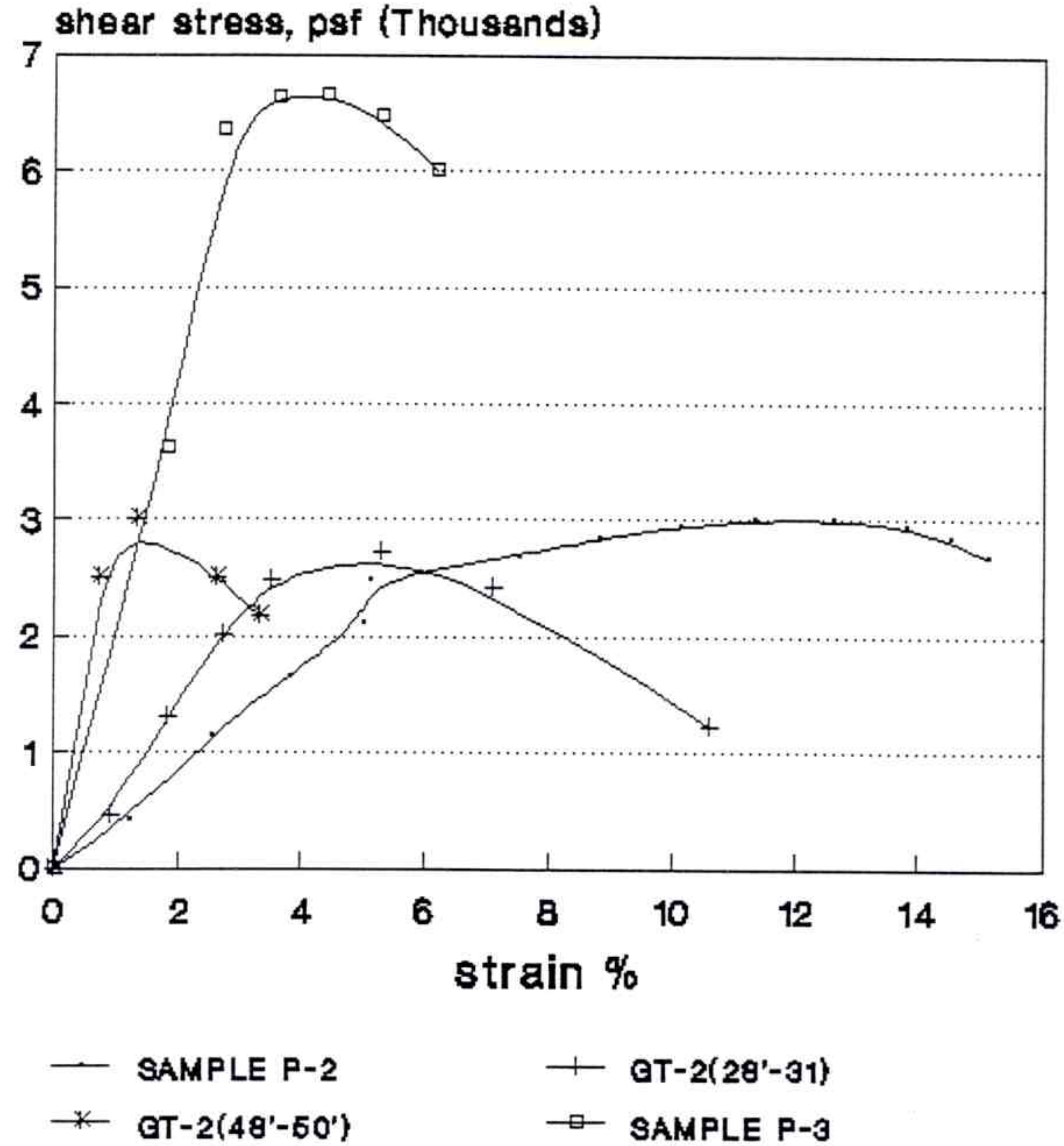
UNCONFINED COMPRESSIVE STRENGTHS



CARIBBEAN SOIL TESTING CO., INC.

PROTECO

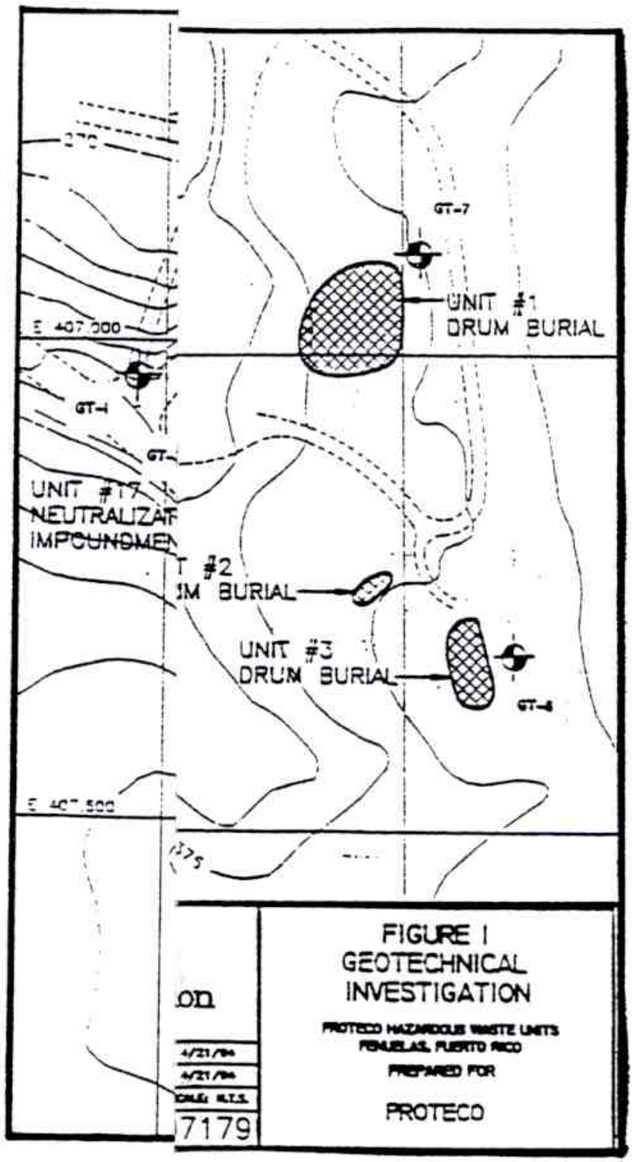
UNCONFINED COMPRESSIVE STRENGTHS



CARIBBEAN SOIL TESTING CO., INC.

FIGURES

Caribbean Soil Testing Co., Inc.
Soils and Materials Testing Laboratory
Tel. (809) 753-0147 / Fax. 759-7880



CARIBBEAN SOIL TESTING CO., INC.

SOILS AND MATERIALS TESTING LABORATORY

258 chile st. hato rey, puerto rico 00917 tels. (809) 753-0147 & 759-7680

PROJECT No: 6373 – 94	APPROVED: AUC

PROTECO
PENUELAS, PUERTO RICO

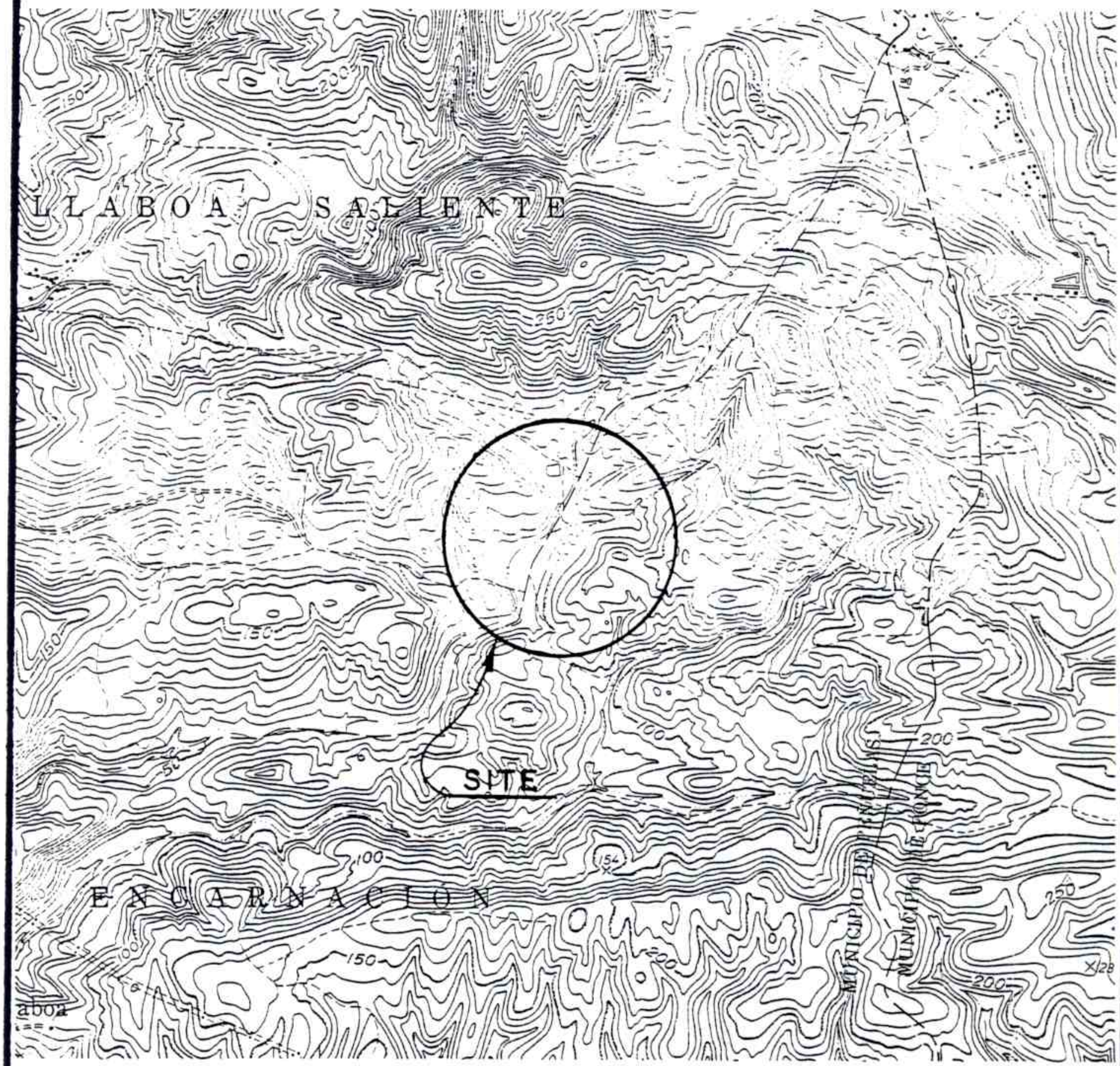


FIGURE -2

CARIBBEAN SOIL TESTING CO., INC.
SOILS AND MATERIALS TESTING LABORATURY

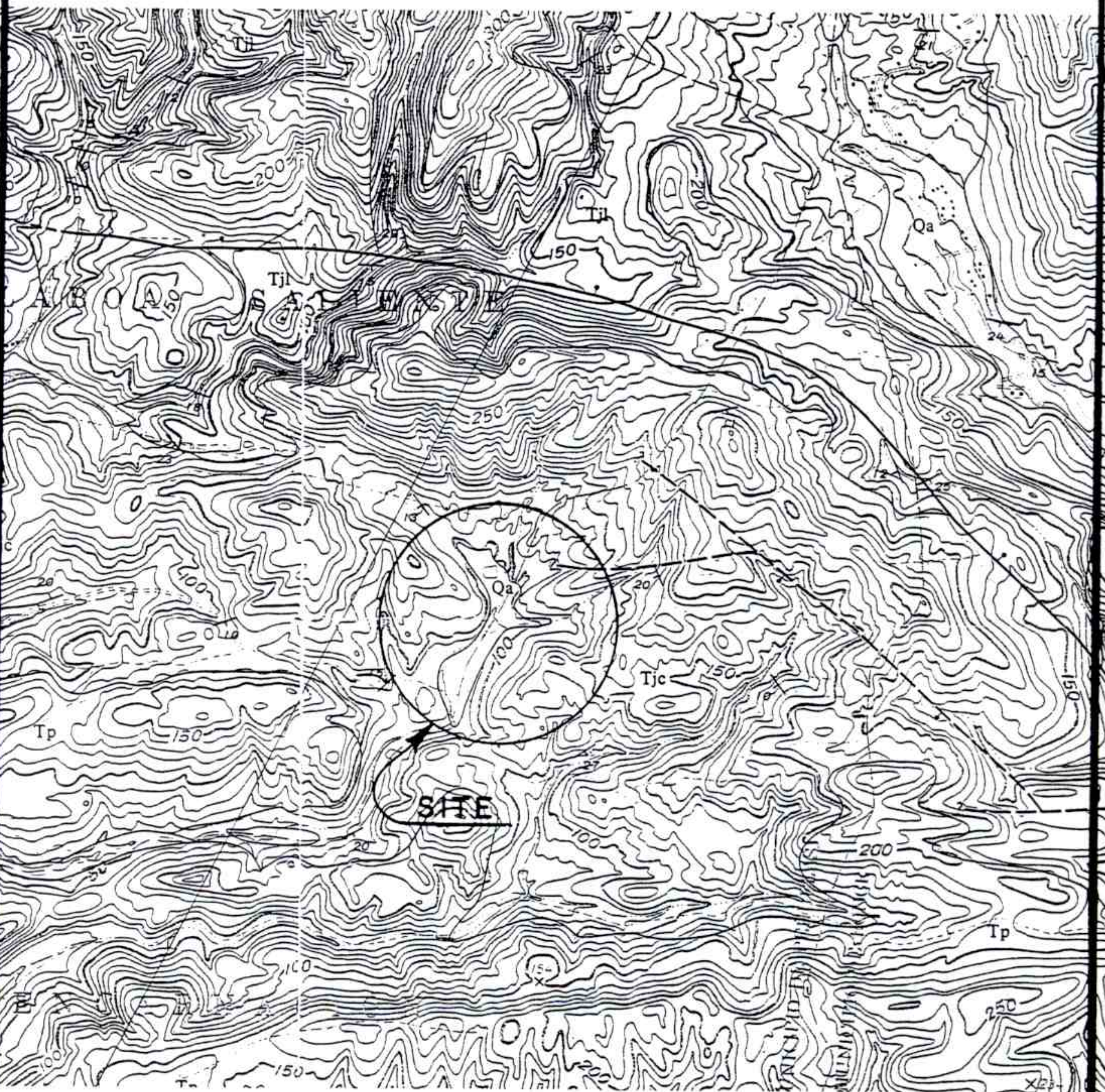


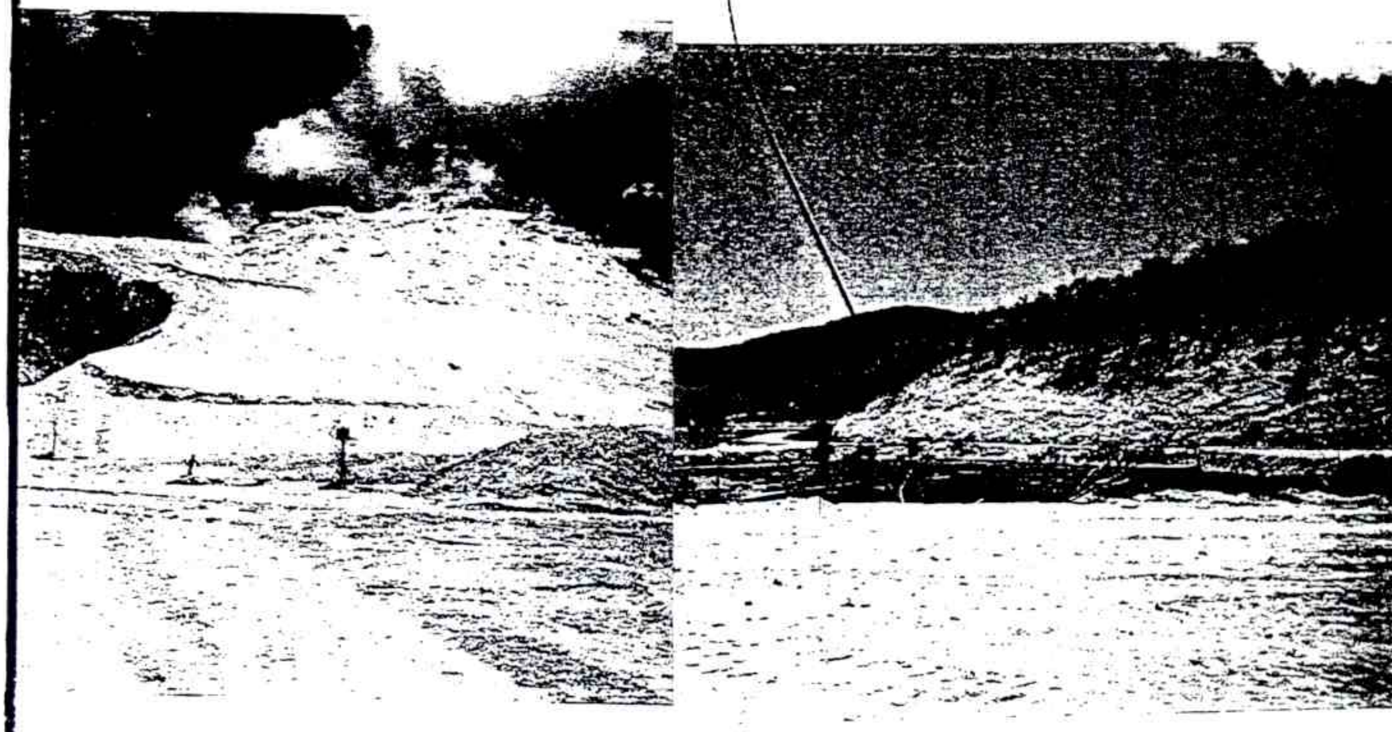
FIGURE-3

CARIBBEAN SOIL TESTING CO., INC.

SOILS AND MATERIALS TESTING LABORATORY

P L A T E S

EXISTING CUT SLOPES WITH
INCLINATION OF ABOUT 45°



), PEÑUELAS ,P.R.

BBEAN SOIL TESTING CO.,INC.

LS AND MATERIALS TESTING LABORATORY

st hato rey, puerto rico 00917 tels (809) 753-0147 & 759-7880

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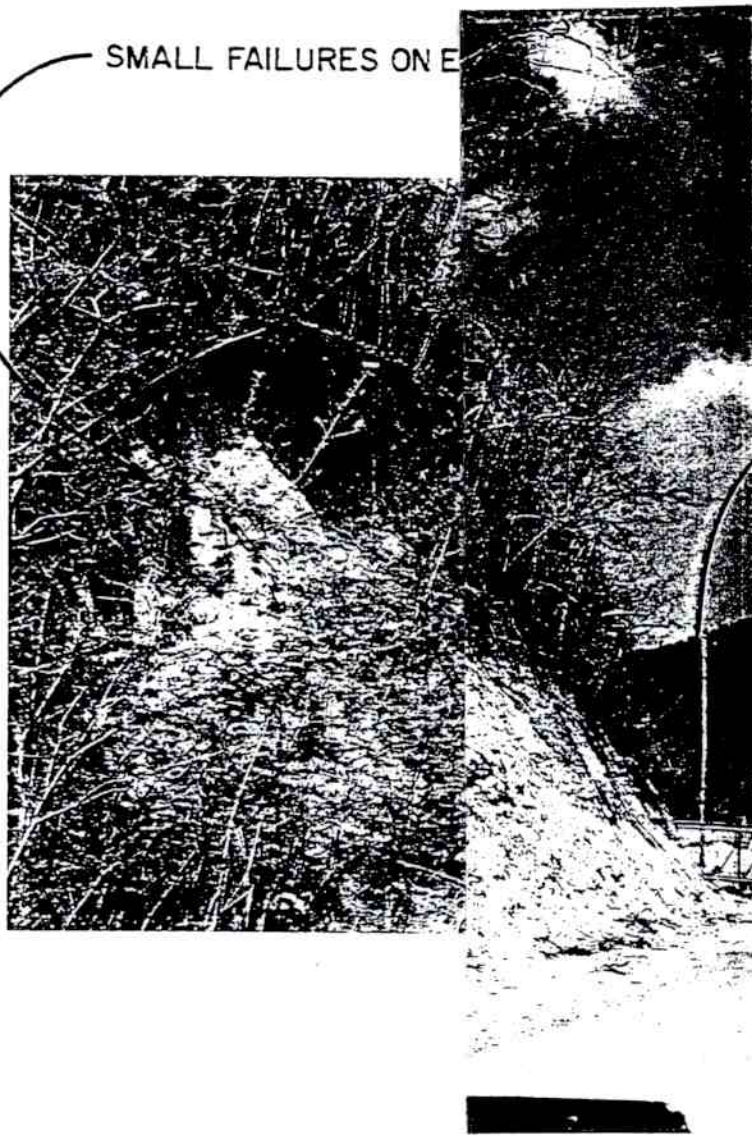


TECO , PEÑUELAS, P.R.

IBBEAN SOIL TESTING CO., INC.

OILS AND MATERIALS TESTING LABORATORY

ic st hoto rey, puerto rico 00917 tels (809) 753-0147 & 759-7880

NO. 6373-94

EXISTING CUT SLOPES ON THE CHALK MEMBER FORMATION

BEDDING PLANES

PROTECO, PEÑUELAS, P. R.

PLATE-B2

CARIBBEAN SOIL TESTING CO., INC.

SOILS AND MATERIALS TESTING LABORATURY

6373-94

APPENDIX G

STABILITY OF NATURAL AND CUT SLOPE ADJACENT TO BURIED WASTE UNITS AT PROTECO'S WASTE DISPOSAL SITE PENUELAS, PUERTO RICO

GEOLOGICAL AND ENVIRONMENTAL SERVICES

GEOLOGICAL ENGINEERING
AND ENVIRONMENTAL SERVICES

GEOLOGICAL ENGINEERING EVALUATION

STABILITY OF NATURAL AND CUT-SLOPE
ADYACENT TO BURIED WASTE UNITS
AT PROTECO'S DISPOSAL SITE
PEÑUELAS, PUERTO RICO

BY:
MARIO SORIANO RESSY
GEOLOGICAL ENGINEER
LIC. 4131

SEPTEMBER ~ 1994

GEOLOGICAL ENGINEERING EVALUATION

***STABILITY OF NATURAL AND CUT-SLOPE
ADYACENT TO BURIED WASTE UNITS
AT PROTECO'S DISPOSAL SITE
PEÑUELAS, PUERTO RICO***

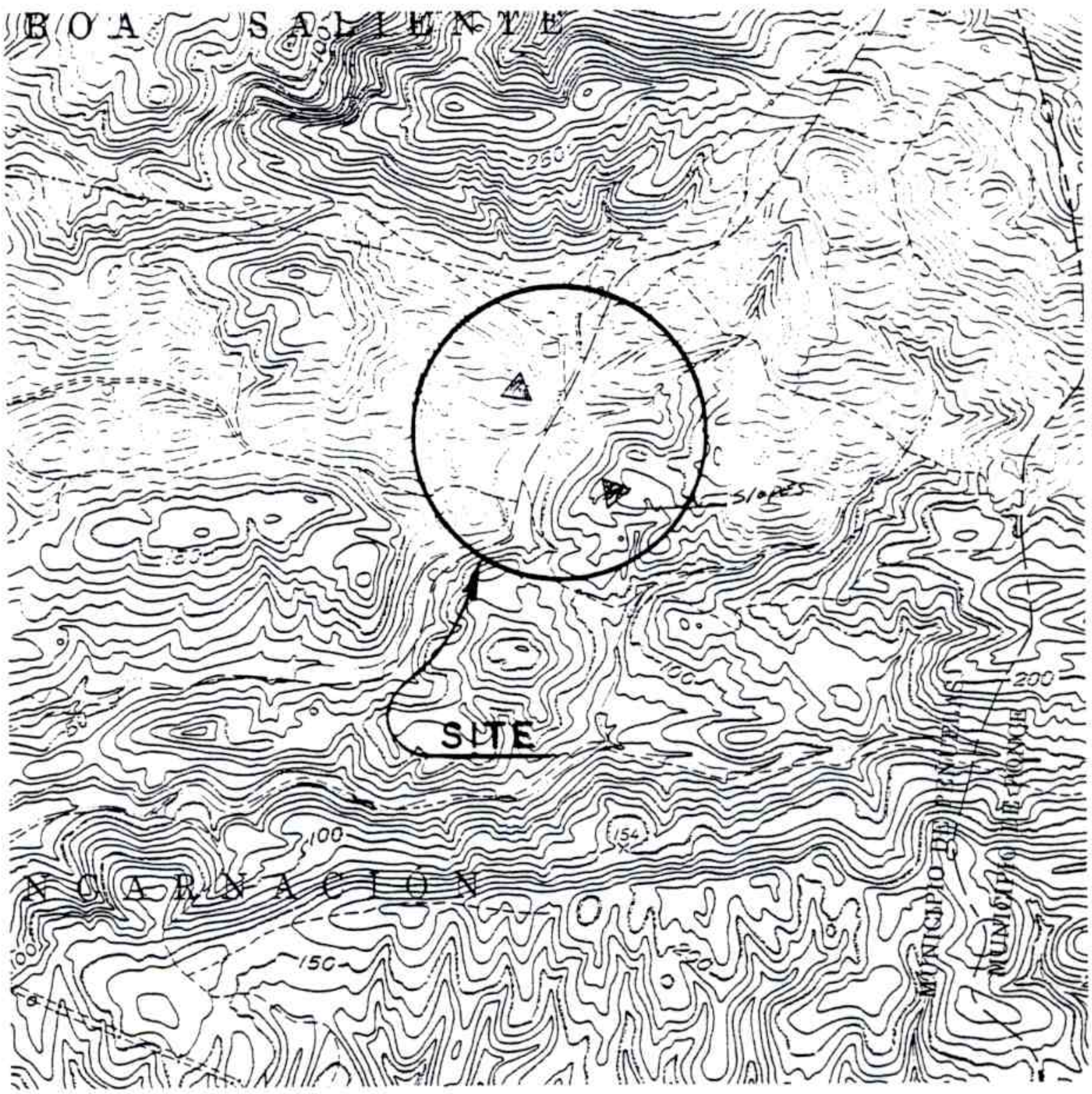
INTRODUCTION:

Responding to a recent request by PROTECO, Inc. the present geological engineering evaluation is extended in relation to the lithic competence and stability of natural and cut-slope existing adyacent to waste units, in the process of being closed, at the PROTECO's facilities in Peñuelas, Puerto Rico.

Pertinent field and office investigation were conducted during the early days of september 1994. The evaluation of both, recent and earlier subsurface explorations thru the boring programs conducted at, and close to this premises, provided valuable information in the determinations contained herein.

Geological Engineering and Environmental Services, gained ample practical technical knowledge of the hydrogeologic and geo-structural characteristic prevailing at said facilities during the six (6) consecutive years that this firm spent as hydrogeological and compliance consultant for PROTECO.

This study concentrated in evaluating the drainage and the



LOCATION PLAN

SOURCE
USGS Peñuelas Quad.

SCALE 1:20,000

Figure number 1

topographically superior terrain that fringes the eastern side of closed waste facilities numbered 12, 13, 16 and 17. It also address cut-slope stability along said units and at the periphery of an existing drainage ditch located inmediately west of waste units numbered 9 and 12. See topographic and location plan in figure number _1_ and _2_.

This report provides basic information that should help in dissipating fears or concern regarding potential massive sliding and/or the interruption of the surface drainage system implemmented along the proximities of the referred closed facilities.

GEOLOGY AT THE SITE:

The dominant geologic material prevailing at the calcareous tertiary hills, located inmediately to the east and west of waste units being closed, is a rather pure chalk member of the Juana Díaz Formation. Both, hard bedded, chalky limestone and massive but poorly stratified chalk material are exposed along cut-slopes created approximately 15 years ago as illustrated in figure number _3_.

The chalky units exhibit an east-west strike or direction with a rather favorable dip or inclination of beds varying between 10 and 15 degrees south. Thus, the inclination of the beds in chalky limestone is almost parallel to the entrenched valley and also to the waste unit facilities emplaced within it. No cut-slope intercepts the chalky beds perpendicularly or in a manner that may impair their stability.

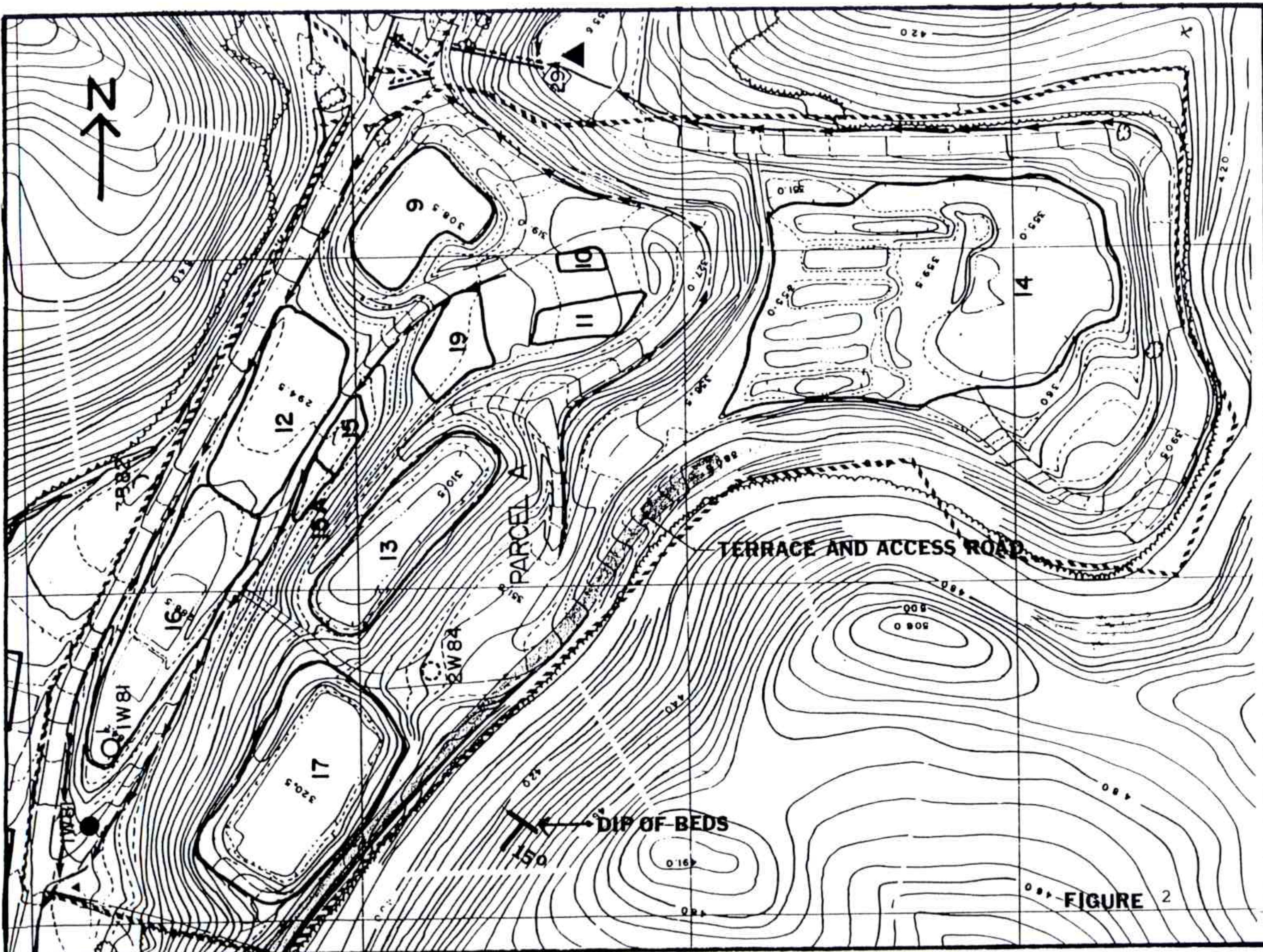


FIGURE 2

TOPOGRAPHIC SETTING AND LOCATION OF FACILITIES

THE 15 YEARD OLD CUT-SLOPE PRESENTED IN THIS PHOTOGRAPHS BORDERS THE DRAINAGE AND ACCESS TERRACE ADYACENT TO FACILITIES NUMBERED 13 AND 17. THE WHITISH AREA WHERE RECENTLY DISTURBED BY HEAVY EQUIPMENT. DARK SURFACE IS INDICATION OF LONG EXPOSURES AND RE-CRYSTALLIZATION.

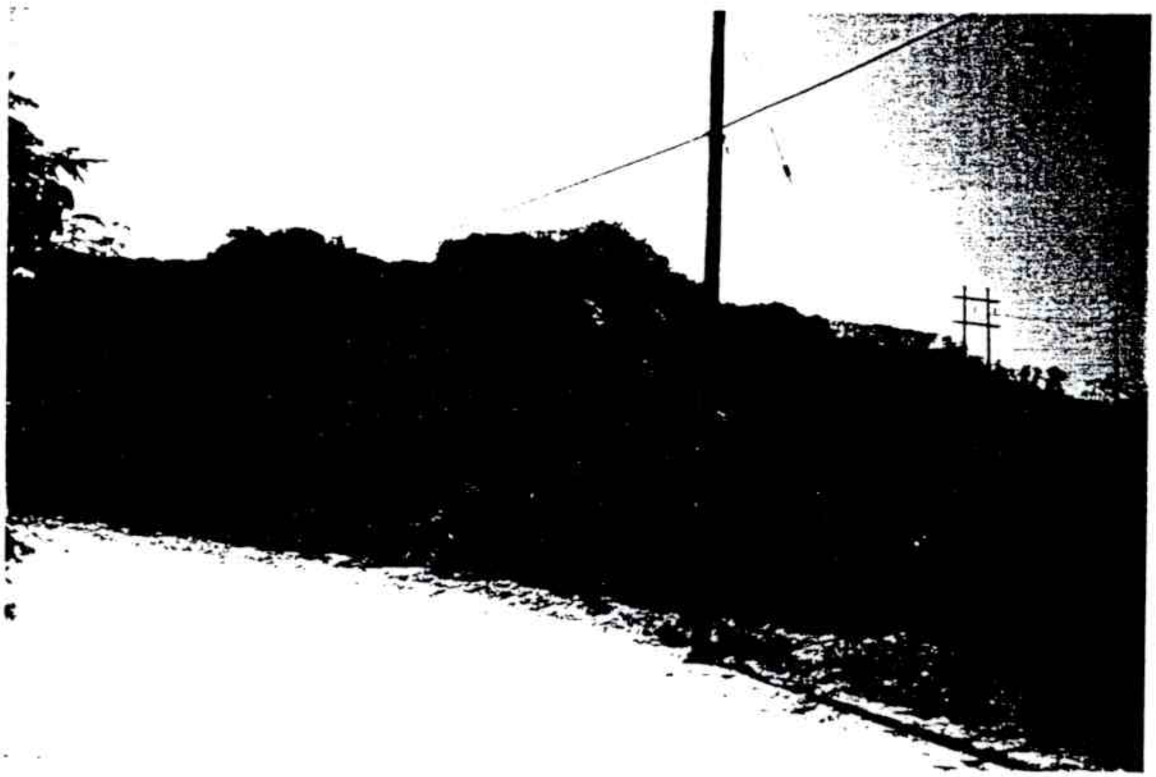
FIGURE NUM. 3 .

The surface of the chalky limestone hills is, on the other hand, capped by a rather hard carapace of case-hardened or recrystallized calcium carbonate surface which in places is almost 3 feet thick. This recrystallization is due to a process in which the calcium carbonate is dissolved and re-precipitated in the upper horizon of the geologic unit, providing a layer which is resistant to erosion and infiltration.

The semi-arid region of Peñuelas where the facilities are located receives between 35 to 40 inches of rainfall yearly. The adyacent topographically superior terrain bordering the referred facilities does not constitute a basin or watershed capable of generating any significant surface flow. A significant amount of humidity, during rainy periods, is retained by the rough carapace and pitted calcareous surface which is readily eliminated by the high evapo-transpiration process. The evapo-transpiration process in this particular region, it has been determined, is twice a great as the rainfall. Consequently, no perennial flow prevail at PROTECO's side. Even intermittent flows are rare or of short durations during the rainy periods.

GEOLOGICAL ENGINEERING CHARACTERISTIC OF THE CHALK AND CHALKY LIMESTONE UNIT:

In general, the geologic material that occupies the superior terrain inmediately east of the facilities under a closure program, is a rather pure chalk, homogenous, and endurated. Even the medium and thick bedded exposures exhibit an excellent stability even in vertical cuts over 10 meters in height.



VIEW ILLUSTRATING ALMOST VERTICAL CUTS CREATED ON QUARRY SITE CLOSE TO PROTECO'S FACILITIES SOME 25 YEARS AGO. NO MASS WASTAGE MOVEMENT OR UNSTABILITY CONDITIONS WAS EVIDENCED ALONG CUT FACES. TINTED BLACK SURFACES INDICATES OLD EXPOSED CUTS WHERE RE-CRYSTALLIZATION OF CALCIUM CARBONATES IS OCCURRING.

FIGURE NUM. 4 .

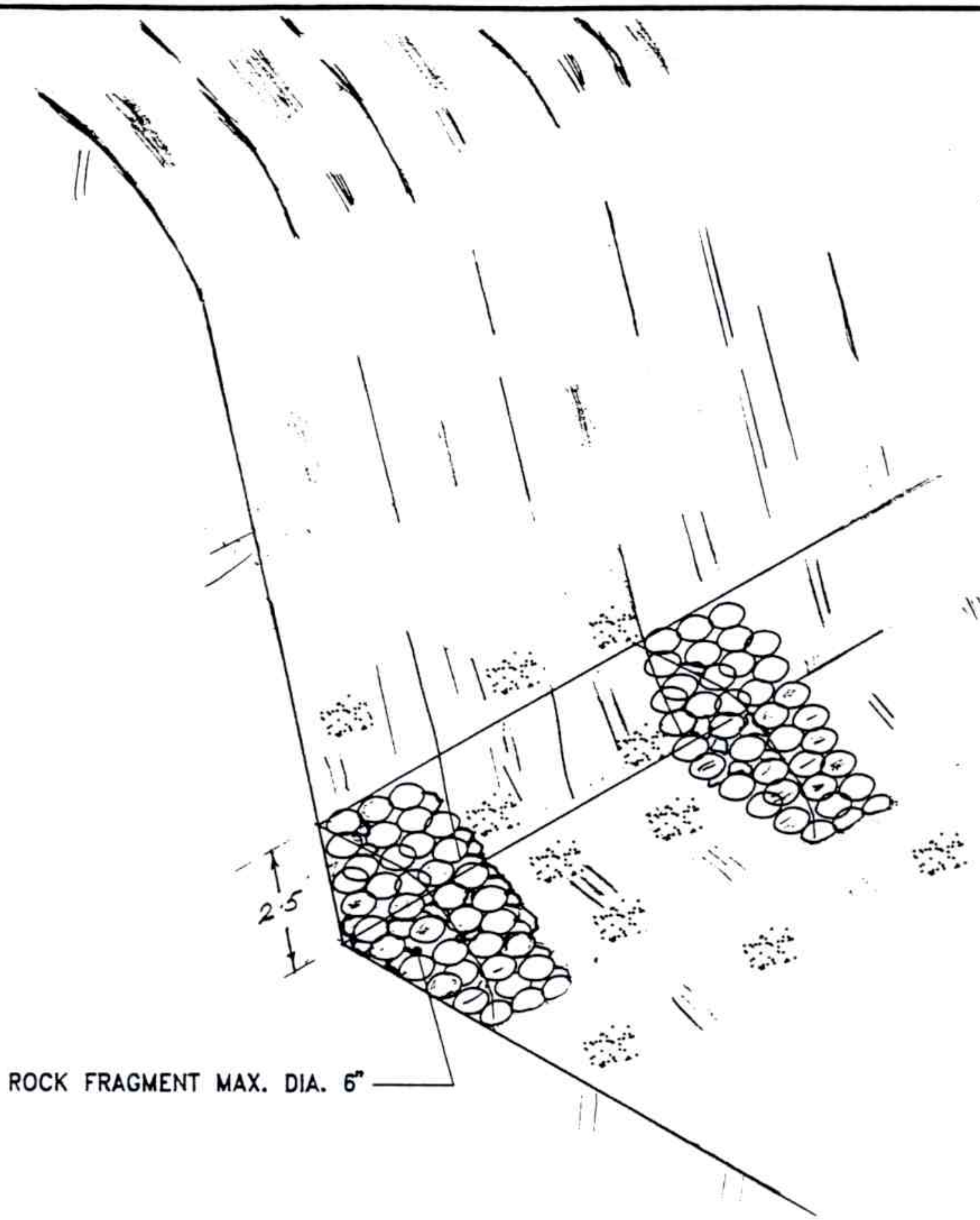
Due to the high impermeability and the good keying actions that exists between the grain in this geologic material, as well as the insufficient hydrogeological conditions for the creation of hydrostatic pressure, massive landsliding is a very rare occurrence in this chalky limestone country of southern Puerto rico.

Vertical and almost vertical slopes both natural and man made are common in the semi-arid limestone country of the Ponce - Peñuelas region where no high water table or perched water table conditions exits within the limestone or calcareous outcrops.

The fringes of the vertical cut-slope bordering the upper northern limits of facilities numbered 13 and 17 were found rather safe and stable 15 years after they were created. No tension cracks or scars of major or minor slidings were found. Only the scar of minor slumps or rockfall were noticed at very few points along the fringes of the cuts. No tension cracks, scarp line or evidence of major incipient unstability was evidenced during field evaluation at the terraces or intervened surfaces along the surrounding hillside.

It is in order to point out that similar but higher cut-slopes created over 25 years ago at a quarry site located along the road leading to PROTECO's facility, exhibit sound stability and competence after all this years. See figure number 4 .

Due to the massiveness and favorable dip angle of this chalk beds, as well as the absence of adverse hydrogeological conditions, this tertiary calcareous material may withstand high vertical slopes for millennium. Such cases can be evidenced in similar



INVERTED OR EXPOSED FRENCH DRAIN TO BE LAID ALONG BASE OF CUT-SLOPE

FIGURE 6

Due to the high impermeability and the good keying actions that exists between the grain in this geologic material, as well as the insufficient hydrogeological conditions for the creation of hydrostatic pressure, massive landsliding is a very rare occurrence in this chalky limestone country of southern Puerto rico.

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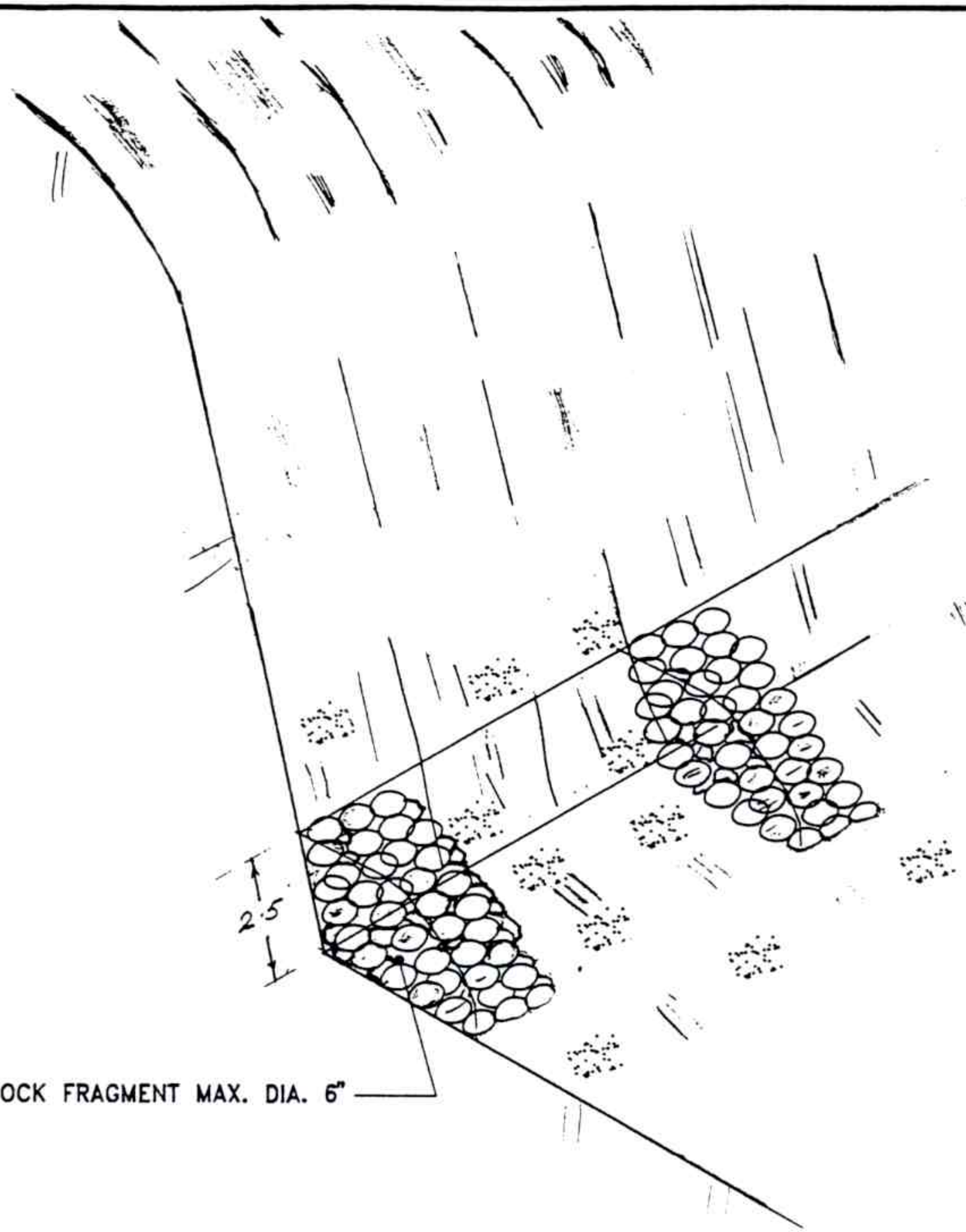

INVERTED OR EXPOSED FRENCH DRAIN TO BE LAID ALONG BASE OF CUT-SLOPE

FIGURE 6

VIEW ILUSTRATING CUT-SLOPE BORDERING A LOW HILL OF BEDDED CHALK AND MAIN DRAINAGE DITCH WITHIN THE FACILITY. THE STABILITY OF THIS CUT-SLOPE IS ALSO CERTIFIED AND THE POSSIBILITY OF MASSIVE SLIDE MOVEMENT IS HEREBY DISCQRDED.

FIGURA NUM. 5 .

chalk outcrops, both, in the highly humid northern calcareous belt, as well as in the drier southern calcareous belt of Puerto Rico, where natural chalky cliffs of over 60 meters in height are very common.

Deterioration and unstability of cut-slope created in these chalk outcrops, can only be attained in the presence of the following factors; a) significant interstratification of expansive clayey material; b) high dip angles of stratified beds; c) high infiltration of humidity and; d) the presence of unfavorable orientation of undermining cut-slopes. Such is not the case at the site of concern!. Here the exposed outcrops consists of pure chalk, the infiltration is scant and the beds dip favorably southward - not perpendicular to the cut. Consequently, sliding, triggered by undermining or along weak interstratified planes, shouldn't be expected to develope in the absence of such adverse factors.

The same favorable stratigraphic and hydrogeological conditions exits for the cut-slope existing across the entrenched valley and bordering a drainage channel opposite the oil laggon. (See figure number 5).

It should be mentioned that soon after cuts on this calcareous geologic meterial are made and exposed to the atmosphere, a recrystallization phenomenon begins to take place. This formation of new mineral grains creates a hardened, protective surface that enhances impermeability and restrict surface erosion. Such exposed surfaces become coated with a black veneer during this *process as can be appreciated in figures* __3__ *and* __4__.

CONCLUSIONS AND RECOMMENDATIONS

1. As evidenced by boring evaluations, field inspection of geologic setting at subject cut-slopes and our work and experience in similar geologic conditions, no massive landsliding by undermining or hydrological adverse conditions can develope at the site of reference.

2. As evidenced by field inspection of the geologic setting prevailing at cut-slopes and surrounding facilities, and taking into consideration subsurface exploration logs of site, it can be safely concluded that no massive landsliding or significant lateral crustal movement should be expected from higher grounds bordering PROTECO's closed facilities. It is emphasized that favorable hydrological and geo-structural conditions prevailing throughout also upholds such conclusions.

3. The upper terrace bordering the facilities along the eastern edge, which serves both, as a drainage outlet and access road, is rather ample, well inclined and structured to favorably channel and dispose of all surface run-off originating at the upper reaches.

To avoid the obstruction of said surface drainage platform along the base of the terrace by slumped geological debris and the overtopping of banks or benches by obstructed waters the following recommendations are extended:

a) *The base of the terrace that serves as drainage outlet should be cleared of existing loose geological debris.*

b) *A wedge of loose angular rock fragments up to 6" in diameter should be placed along the entire base and against existing slope as illustrated in figure number* _6_.

c) *Such wedge of rock fragments should be emplaced as an inverted "french drain" capable of allowing the free flow of suface drainage along the base of the cut-slope and thru the rock voids even if a slump of geologic material falls from the upper edges over it. The wedge will allow for sufficient passages or interstices to allow free uninterrupted lateral drainage even during extraordinary rain falls. Periodic maintenance thru the elimination of wastage debris will provide continuous effectiveness of drainage outlet.*

d) *The wedge of rock fragments will also provides lateral support at base of cut and check scouring due to velocity of flow.*

Submitted on September 19th, 1994.

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E AGO 98
PUERTO RICO

MARIO SORIANO RESSY

APPENDIX H

SLOPE STABILITY CALCULATIONS

SLOPE STABILITY ANALYSIS
PROTECO LANDFILL
PUNEALUS, PUERTO RICO
OHM PROJECT #16139

GENERAL

OHM Remediation Services Corp. (OHM) is performing work at the Proteco Landfill which includes construction of a final cover and other earthwork. Construction of the final cover will include changing the inclination of several slopes at the site. The short term and long term stability of these proposed slopes was evaluated using the REAME computer model. The REAME computer model was written by Dr. Yang H. Huang, P.E., and published in March 1994 by Civil Engineering Software Center, College of Engineering, University of Kentucky, Lexington, Kentucky. The model analysis is performed on two-dimensional circular slip surfaces utilizing six different methods including, normal, simplified Bishop, Spencer, Modified Spencer (Morgenstern and Price), and two versions of Janbu. This paper provides an explanation of how input parameters for this model were selected.

LABORATORY TESTING

Six geotechnical borings were performed at the site in the spring of 1994. Boring locations and depths were selected by OHM and performed by Caribbean Soil Testing Company, Inc. The borings were performed to depths ranging from 15 to 50 feet below the prevailing ground surface. Standard Penetration Testing (SPT) as provided for in ASTM D-1586 was performed at select intervals as the borings were advanced. Undisturbed soil samples were also collected at select locations using a Denison Soil Sampler (ASTM D-1452), and shelby tubes.

Nine of the soil samples collected from the borings were submitted for Atterberg Limits Testing (ASTM D-4386), four samples were submitted for unconfined compression testing (ASTM D-2166), three samples were submitted for grain size analysis (ASTM D-422), and one sample was submitted for hydraulic conductivity testing (ASTM D-5086). The test results are provided in the report submitted by Caribbean Soil Testing Company, Inc. to OHM.

Five bulk soil samples were also collected at the site. These bulk samples are representative of material proposed for use in the the low permeability layer of the final cover. Five of the bulk samples were submitted for Standard Proctor (ASTM D-698), four for Atterberg Limits Testing (ASTM D-4386) and hydraulic conductivity (ASTM D-5086), three for unconfined compression (ASTM D-2166), and one for grain size analysis (ASTM D-422). The test results are provided in the report submitted by Caribbean Soil Testing Company, Inc. to OHM.

SOIL CONDITIONS

In general, soils at the site were found to have relatively uniform physical characteristics. The soil consists of a clay (CL to CH by USCS classification) with liquid limits ranging from 38.5 % to 75.5 % and plastic index (PI) ranging from 22.5 % to 48.0 %. SPT blow counts ranged from 24 to greater than 100 blows per foot. Undisturbed samples failed at 830 to

3,000 psf in unconfined compression. Samples with a higher PI sustained a higher load in unconfined compression. The clay contains very little sand (32% sand at a LL of 38.5 and PI of 22.5, 3% sand at a LL of 70 % and PI of 35 %).

Auger refusal (i.e. bedrock) was encountered at 45 feet below the ground surface in one boring. Bedrock in the area consists of interbedded limestone and chalk. Groundwater was not encountered in any of the borings.

SELECTION OF MODEL PARAMETERS

OHM developed a site plan with contours of the existing ground surface and contours of the proposed final cover surface. Proposed slopes have a maximum inclination of 3 horizontal to 1 vertical (3:1). The tallest slope was selected for detailed slope stability analysis. This slope is located on the northwest side of waste unit No. 17, has a slope of roughly 3.3:1, and a height of roughly 30 feet. The slope and cross section selected for analysis are indicated on Figure 1 and Figure 2.

Subsurface materials in the cross section were segregated into three layers, rock, residual soils, and cover soils. The upper surface of bedrock is assumed to occur at elevation 260 across the entire cross section. The material from rock surface to the existing ground surface is residual soil. Cover materials will consist of residual soils that have been processed and compacted.

Physical properties of residual soils and cover materials are derived from the Caribbean Soil Testing Company, Inc. geotechnical report and corroborating published data for similar soils. The corroborating data was obtained from *Stability Analysis of Earth Slopes*, Yang H. Huang, Van Nostrand Reinhold Company, New York, 1983 and *Introductory Soil Mechanics and Foundations Third Edition*, George B. Sowers and George F. Sowers, The MacMillian Company, London, 1970.

A dry residual soil density of 90 pcf was selected based on SPT blow counts and a CL-CH material. Natural moisture of residual soils was assumed to be equal to the plastic limit (25 %), resulting in a wet density of 112.5 pcf. Fill materials in the final cover were assumed to have a compacted dry density of 90 pcf, a moisture content of 27 %, and a wet density of 114.3 pcf based on Standard Proctor test results.

Two scenarios (short term and long term) were selected for slope stability analysis. The short term scenario provided for a case where soil strength was derived entirely from cohesion. Soil strength in the long term case was derived from both cohesion and internal friction.

SHORT TERM SOIL PARAMETERS

A residual soil cohesion of 1,250 psf was selected based on results of unconfined compression of an undisturbed sample collected from GT-2. Fill material cohesion of 1,350 psf was selected based on unconfined compression performed on remolded samples.

LONG TERM SOIL PARAMETERS

An effective friction angle of 25° and cohesion of 100 psf were selected for residual soils based on the Atterberg Limits, USCS classification, and Table 3.1 and Figures 3.10 and 3.11 in Huang (1983). An effective friction angle of 20° and cohesion of 200 psf were selected for final cover fill materials based on the Atterberg Limits, USCS classification, and Table 3.1 and Figures 3.10 and 3.11 in Huang (1983).

FAILURE ZONES

Two types of slope failures are anticipated based on the vertical profile of the slope. The entire slope could fail with a failure surface through the residual soils, or failure could occur in the final cover compacted fill material. The variable NRCS (Number of Radius Control Zones) in the REAME program was utilized to control the locations of two failure circles. One failure circle was forced to be located in the residual soil (between bedrock and the existing ground surface), and one failure circle was forced to occur in the new cap fill material (between the existing ground surface and proposed final cover ground surface).

MODEL RESULTS

The lowest short term factor of safety was associated with a deep seated failure surface that passed through residual soils to the rock surface. A 2.03 factor of safety was calculated for this case. A 2.64 factor of safety was calculated for failure in the proposed final cover fill materials.

The lowest long term factor of safety was associated with a classical failure surface that started about 10 to 20 feet from the slope crest, passed through residual soils, and daylighted at the slope toe. A 1.83 factor of safety was calculated for this case. This was the lowest factor of safety calculated. A 3.46 factor of safety was calculated for long term failure in the proposed final cover fill materials.

Table 3.1 Average Effective Shear Strength of Compacted Soils.

UNIFIED CLASSIFICATION	SOIL TYPE	PROCTOR COMPACTION MAXIMUM DRY DENSITY pcf	PROCTOR COMPACTION OPTIMUM MOISTURE CONTENT %	AS COMPACTED COHESION c_o tsf	SATURATED COHESION c_{sat} tsf	FRICTION ANGLE $\bar{\phi}$ deg
GW	well graded clean gravels, gravel-sand mixture	>119	<13.3	*	*	>38
GP	poorly graded clean gravels, gravel sand mixture	>110	<12.4	*	*	>37
GM	silty gravels, poorly graded gravel-sand-silt	>114	<14.5	*	*	>34
GC	clayey gravels, poorly graded gravel-sand-clay	>115	<14.7	*	*	>31
SW	well graded clean sands, gravelly sands	119±5	13.3±2.5	0.41±0.04	*	38±1
SP	poorly graded clean sands, sand-gravel mixture	110±2	12.4±1.0	0.24±0.06	*	37±1
SM	silty sands, poorly graded sand-silt mixture	114±1	14.5±0.4	0.53±0.06	0.21±0.07	34±1
SM-SC	sand-silt-clay with slightly plastic fines	119±1	12.8±0.5	0.21±0.07	0.15±0.06	33±3
SC	clayey sands, poorly graded sand-clay mixture	115±1	14.7±0.4	0.78±0.16	0.12±0.06	31±3
ML	inorganic silts and clayed silts	103±1	19.2±0.7	0.70±0.10	0.09±*	32±2
ML-CL	mixtures of inorganic silts and clays	109±2	16.8±0.7	0.66±0.18	0.23±*	32±2
CL	inorganic clays of low to medium plasticity	108±1	17.3±3	0.91±0.11	0.14±0.02	28±2
OL	organic silts and silty clays of low plasticity	*	*	*	*	*
MH	inorganic clayey silts, elastic silts	82±4	36.3±3.2	0.76±0.31	0.21±0.09	25±3
CH	inorganic clays of high plasticity	94±2	25.5±1.2	1.07±0.35	0.12±0.06	19±5
OH	organic clays and silty clays	*	*	*	*	*

*denotes insufficient data, > is greater than, < is less than
(After Bureau of Reclamation, 1973; 1 pcf=157.1 N/m³, 1 tsf=95.8 kPa)

The shear strength listed in Table 3.1 is for compacted soils. For natural soils, the effective cohesion may be larger or smaller than the listed values depending on whether the soil is overly or normally consolidated, but the effective angle of internal friction should not be much different. Kenney (1959) presented the relationship between sin $\bar{\phi}$ and the plasticity index for normally consolidated soils, as shown in Figure 3.10. Although there is considerable scatter, a definite trend toward decreasing $\bar{\phi}$ with increasing plasticity is apparent. Bjerrum and Simons (1960) presented a similar relationship for both undisturbed and remolded soil as shown in Fig. 3.11. The relationship by Kenney (1959) is plotted in dashed curve for comparison. Skempton (1964) presented a correlation between the residual effective angle of internal

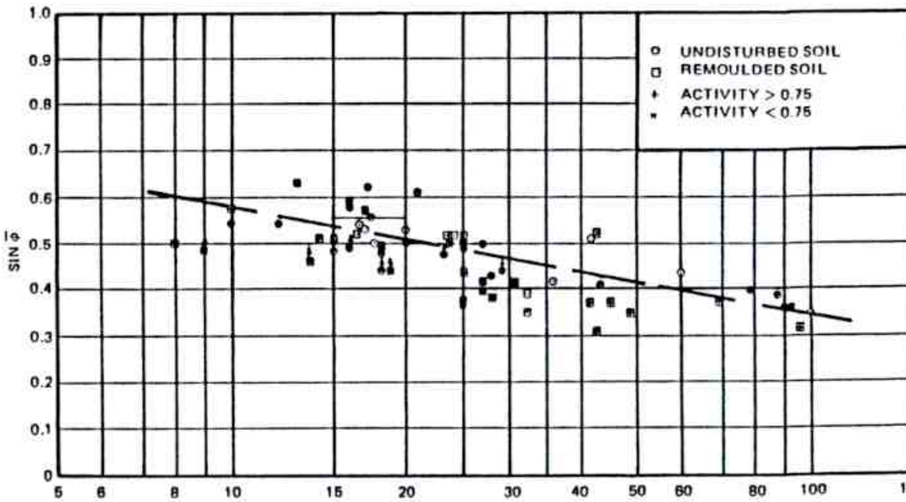
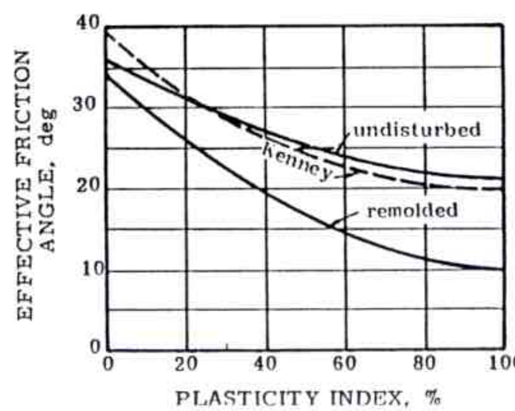


FIGURE 3.10. Plasticity index versus sin$\bar{\phi}$ for normally consolidated soils. (After Kenney, 1959)



FI... 3.11. ...ticity ... versu... ...ctive f... angl... (After B... and ...ons,

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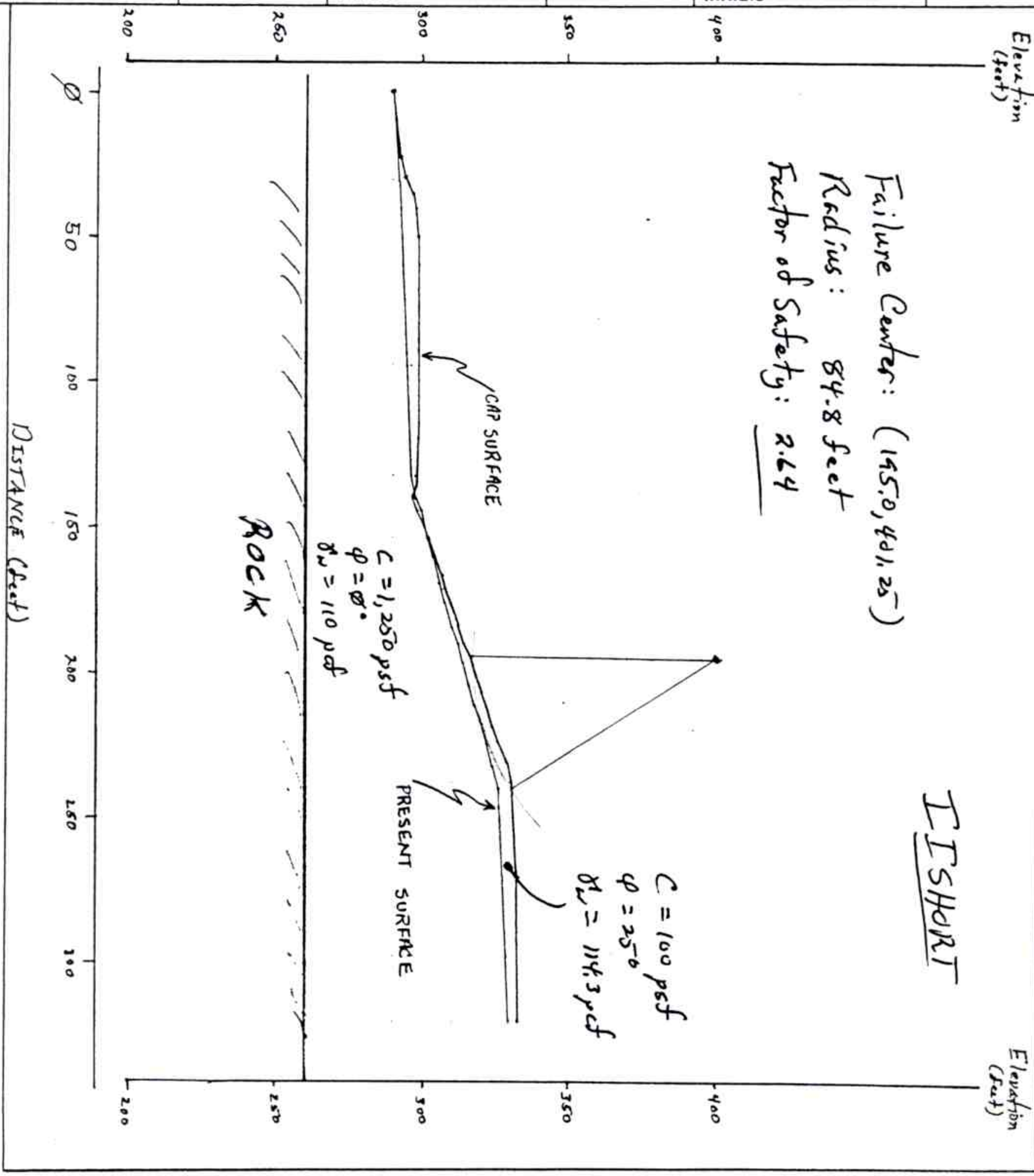
Rev. 08/89

Page ______ of ______					
Proj. No. 16139	Client Proteco	Location Puerto Rico		Subject Slope Stability	
Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date



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UNIVERSITY OF KENTUCKY, LEXINGTON, KY 40506

INPUT FILE NAME -c:\16139\iishort2.dta

TITLE -iishort2.dta

NO. OF STATIC AND SEISMIC CASES- 1

NO. OF NONCIRCULAR SLIP SURFACES= 0


CASE NO. 1    SEISMIC COEFFICIENT= 0

NO. OF BOUNDARY LINES= 3

NO. OF POINTS ON BOUNDARY LINE 1 = 2
 1  X COORD.= 0             Y COORD.= 260
 2  X COORD.= 320           Y COORD.= 260

NO. OF POINTS ON BOUNDARY LINE 2 = 5
 1  X COORD.= 0             Y COORD.= 290
 2  X COORD.= 132           Y COORD.= 296
 3  X COORD.= 140           Y COORD.= 297
 4  X COORD.= 240           Y COORD.= 326
 5  X COORD.= 320           Y COORD.= 330

NO. OF POINTS ON BOUNDARY LINE 3 = 10
 1  X COORD.= 0             Y COORD.= 290
 2  X COORD.= 23            Y COORD.= 292
 3  X COORD.= 40            Y COORD.= 298
 4  X COORD.= 50            Y COORD.= 299
 5  X COORD.= 113           Y COORD.= 299
 6  X COORD.= 136           Y COORD.= 298
 7  X COORD.= 140           Y COORD.= 297
 8  X COORD.= 238           Y COORD.= 330
 9  X COORD.= 270           Y COORD.= 332
 10 X COORD.= 320           Y COORD.= 332

LINE NO. AND SLOPE OF EACH SEGMENT ARE:
 1         +0.000
 2         +0.045     +0.125     +0.290     +0.050
 3         +0.087     +0.353     +0.100     +0.000     -0.043     -0.250
           +0.337     +0.063     +0.000

MIN. DEPTH OF TALLEST SLICE= 0
NO. OF RADIUS CONTROL ZONES= 1

RADIUS DECREMENT FOR ZONE 1 = 0
NO. OF CIRCLES FOR ZONE 1 = 10
ID NO. FOR FIRST CIRCLE FOR ZONE 1 = 1
NO. OF BOTTOM LINES FOR ZONE 1 = 1

FOR ZONE 1     LINE SEQUENCE 1
LINE NO.= 2    BEG. NO.= 1    END NO.= 5
UNIT WEIGHT OF WATER= 62.4
```

SOIL NO.	COHESION	FRIC. ANGLE	UNIT WEIGHT
1	1250	0	110
2	100	25	114.3

NO SEEPAGE
USE SEARCH
NO. OF SLICES= 40
NO. OF ADD. RADII= 5
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF CENTERS TO BE ANLYZED= 1

ONLY F. S. AT EACH CENTER WILL BE PRINTED
SLICES WILL BE SUBDIVIDED

SEARCH STARTED AT CENTER NO. 1
X COORDINATE= 200 Y COORDINATE= 400
X INCREMENT = 5 Y INCREMENT = 5

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
200	400	10	1	82.213	2.668	0
205	400	10	1	80.820	2.784	0
195	400	10	1	83.605	2.635	0
190	400	10	1	84.998	2.700	0
195	405	10	1	88.408	2.638	0
195	395	10	1	78.803	2.649	0
196.25	400	10	1	83.257	2.635	0
193.75	400	10	1	83.954	2.641	0
195	401.25	10	1	84.806	2.635	0
195	402.5	10	1	86.006	2.635	0
196.25	401.25	10	1	84.458	2.637	0
193.75	401.25	10	1	85.154	2.638	0

AT POINT (195 401.25) RADIUS 84.806

THE MINIMUM FACTOR OF SAFETY IS 2.635

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL SLIP SURFACE

SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	SLICE SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	2	1.086	0.163	0.000	0.040	0.202E+02	0.202E+02	0.100E+05	0.692E+02
2	2	1.086	0.477	0.000	0.053	0.592E+02	0.592E+02	0.116E+05	0.267E+03
3	2	1.086	0.778	0.000	0.066	0.965E+02	0.965E+02	0.130E+05	0.541E+03
4	2	1.086	1.065	0.000	0.079	0.132E+03	0.132E+03	0.144E+05	0.884E+03
5	2	1.086	1.337	0.000	0.092	0.166E+03	0.166E+03	0.158E+05	0.129E+04
6	2	1.086	1.596	0.000	0.104	0.198E+03	0.198E+03	0.170E+05	0.175E+04
7	2	1.086	1.840	0.000	0.117	0.228E+03	0.228E+03	0.182E+05	0.227E+04

8	2	1.086	2.070	0.000	0.130	0.257E+03	0.257E+03	0.194E+05	0.283E+04
9	2	1.086	2.286	0.000	0.143	0.284E+03	0.284E+03	0.204E+05	0.344E+04
10	2	1.086	2.488	0.000	0.156	0.309E+03	0.309E+03	0.214E+05	0.408E+04
11	2	1.086	2.675	0.000	0.168	0.332E+03	0.332E+03	0.223E+05	0.474E+04
12	2	1.086	2.848	0.000	0.181	0.353E+03	0.353E+03	0.231E+05	0.543E+04
13	2	1.086	3.006	0.000	0.194	0.373E+03	0.373E+03	0.239E+05	0.614E+04
14	2	1.086	3.150	0.000	0.207	0.391E+03	0.391E+03	0.245E+05	0.686E+04
15	2	1.086	3.278	0.000	0.220	0.407E+03	0.407E+03	0.251E+05	0.758E+04
16	2	1.086	3.392	0.000	0.232	0.421E+03	0.421E+03	0.257E+05	0.830E+04
17	2	1.086	3.490	0.000	0.245	0.433E+03	0.433E+03	0.261E+05	0.901E+04
18	2	1.086	3.573	0.000	0.258	0.443E+03	0.443E+03	0.265E+05	0.971E+04
19	2	1.086	3.641	0.000	0.271	0.452E+03	0.452E+03	0.268E+05	0.104E+05
20	2	1.086	3.693	0.000	0.284	0.458E+03	0.458E+03	0.270E+05	0.110E+05
21	2	1.086	3.730	0.000	0.296	0.463E+03	0.463E+03	0.271E+05	0.116E+05
22	2	1.086	3.751	0.000	0.309	0.465E+03	0.465E+03	0.272E+05	0.122E+05
23	2	1.086	3.755	0.000	0.322	0.466E+03	0.466E+03	0.272E+05	0.127E+05
24	2	1.086	3.743	0.000	0.335	0.464E+03	0.464E+03	0.271E+05	0.132E+05
25	2	1.086	3.714	0.000	0.348	0.461E+03	0.461E+03	0.269E+05	0.136E+05
26	2	1.086	3.669	0.000	0.360	0.455E+03	0.455E+03	0.267E+05	0.139E+05
27	2	1.086	3.606	0.000	0.373	0.447E+03	0.447E+03	0.263E+05	0.142E+05
28	2	1.086	3.526	0.000	0.386	0.438E+03	0.438E+03	0.259E+05	0.143E+05
29	2	1.086	3.428	0.000	0.399	0.425E+03	0.425E+03	0.255E+05	0.144E+05
30	2	1.086	3.313	0.000	0.412	0.411E+03	0.411E+03	0.249E+05	0.144E+05
31	2	1.086	3.179	0.000	0.425	0.394E+03	0.394E+03	0.243E+05	0.142E+05
32	2	1.086	3.026	0.000	0.437	0.375E+03	0.375E+03	0.236E+05	0.139E+05
33	2	1.086	2.854	0.000	0.450	0.354E+03	0.354E+03	0.228E+05	0.135E+05
34	2	1.086	2.662	0.000	0.463	0.330E+03	0.330E+03	0.220E+05	0.130E+05
35	2	1.086	2.451	0.000	0.476	0.304E+03	0.304E+03	0.210E+05	0.123E+05
36	2	1.086	2.219	0.000	0.489	0.275E+03	0.275E+03	0.201E+05	0.114E+05
37	2	1.029	1.973	0.000	0.501	0.232E+03	0.232E+03	0.180E+05	0.986E+04
38	2	1.142	1.543	0.000	0.514	0.201E+03	0.201E+03	0.181E+05	0.878E+04
39	2	1.086	0.934	0.000	0.527	0.116E+03	0.116E+03	0.147E+05	0.518E+04
40	2	1.086	0.317	0.000	0.540	0.393E+02	0.393E+02	0.122E+05	0.180E+04
							SUM	0.874E+06	0.335E+06

AT CENTER (195.000, 401.250) WITH RADIUS 84.806 AND SEISMIC COEFF. 0.00
FACTOR OF SAFETY BY NORMAL METHOD IS 2.608
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS 2.635

OHM Corporation

COMPUTATION SHEET

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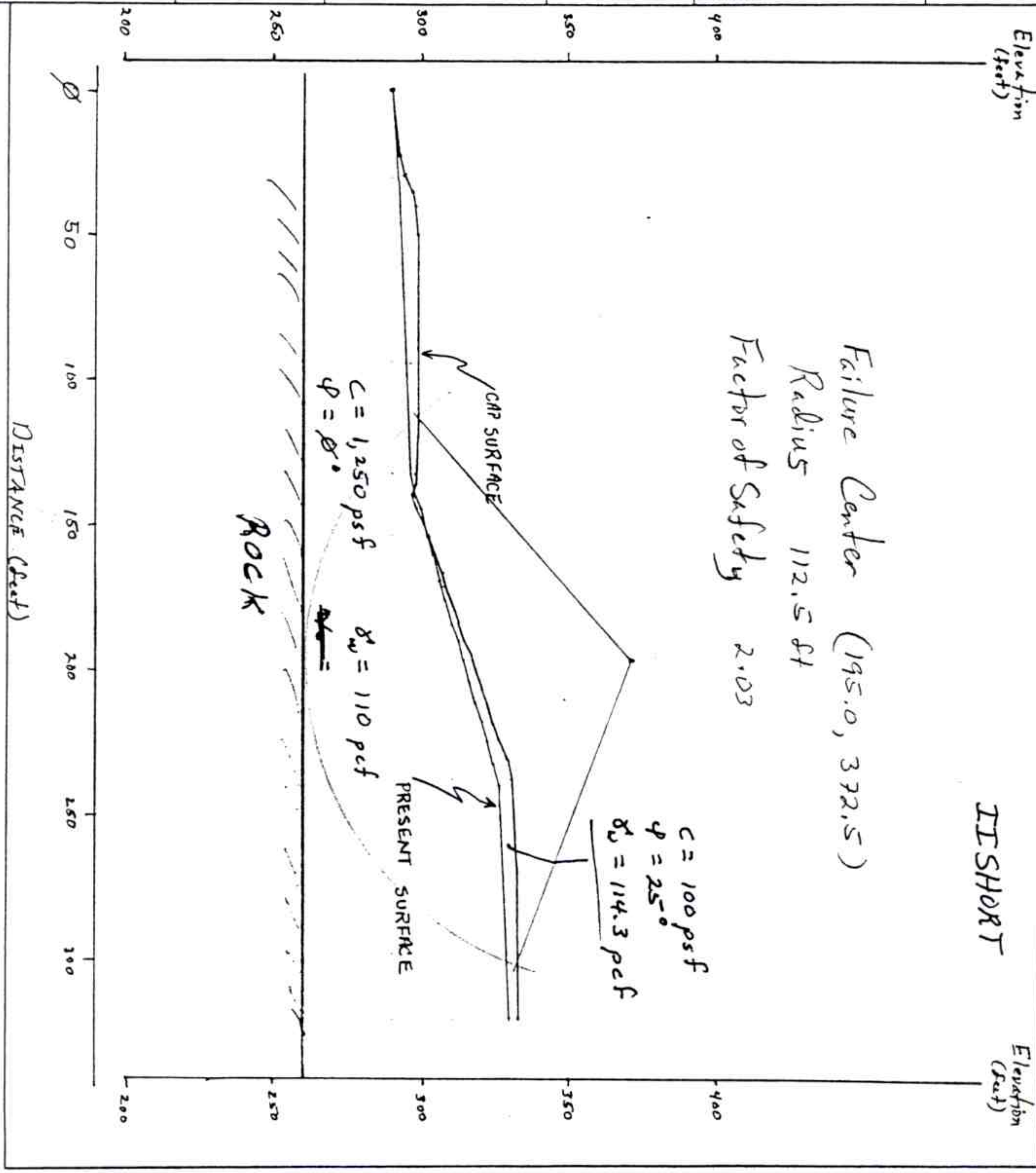
Midwest Tech. Servs.
Rev. 08/89

Page ______ of ______

Proj. No. 16139	Client Proteco	Location Puerto Rico	Subject Slope Stability

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date



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UNIVERSITY OF KENTUCKY, LEXINGTON, KY 40506

INPUT FILE NAME -c:\16139\iishort.dta

TITLE -iishort.dta

NO. OF STATIC AND SEISMIC CASES- 1

NO. OF NONCIRCULAR SLIP SURFACES= 0


CASE NO. 1    SEISMIC COEFFICIENT= 0

NO. OF BOUNDARY LINES= 3

NO. OF POINTS ON BOUNDARY LINE 1 = 2
 1  X COORD.= 0            Y COORD.= 260
 2  X COORD.= 320          Y COORD.= 260

NO. OF POINTS ON BOUNDARY LINE 2 = 5
 1  X COORD.= 0            Y COORD.= 290
 2  X COORD.= 132          Y COORD.= 296
 3  X COORD.= 140          Y COORD.= 297
 4  X COORD.= 240          Y COORD.= 326
 5  X COORD.= 320          Y COORD.= 330

NO. OF POINTS ON BOUNDARY LINE 3 = 10
 1  X COORD.= 0            Y COORD.= 290
 2  X COORD.= 23           Y COORD.= 292
 3  X COORD.= 40           Y COORD.= 298
 4  X COORD.= 50           Y COORD.= 299
 5  X COORD.= 113          Y COORD.= 299
 6  X COORD.= 136          Y COORD.= 298
 7  X COORD.= 140          Y COORD.= 297
 8  X COORD.= 238          Y COORD.= 330
 9  X COORD.= 270          Y COORD.= 332
 10 X COORD.= 320          Y COORD.= 332

LINE NO. AND SLOPE OF EACH SEGMENT ARE:
 1        +0.000
 2        +0.045    +0.125    +0.290    +0.050
 3        +0.087    +0.353    +0.100    +0.000    -0.043    -0.250
          +0.337    +0.063    +0.000

MIN. DEPTH OF TALLEST SLICE= 0
NO. OF RADIUS CONTROL ZONES= 2

RADIUS DECREMENT FOR ZONE 1 = 0
NO. OF CIRCLES FOR ZONE 1 = 10
ID NO. FOR FIRST CIRCLE FOR ZONE 1 = 1
NO. OF BOTTOM LINES FOR ZONE 1 = 1

FOR ZONE 1     LINE SEQUENCE 1
LINE NO.= 1    BEG. NO.= 1   END NO.= 2

RADIUS DECREMENT FOR ZONE 2 = 0
```

NO. OF CIRCLES FOR ZONE 2 = 5
ID NO. FOR FIRST CIRCLE FOR ZONE 2 = 1
NO. OF BOTTOM LINES FOR ZONE 2 = 1

FOR ZONE 2   LINE SEQUENCE 1
LINE NO.= 2   BEG. NO.= 1   END NO.= 5
UNIT WEIGHT OF WATER= 62.4

SOIL NO.	COHESION	FRIC. ANGLE	UNIT WEIGHT
1	1250	0	110
2	100	25	114.3

NO SEEPAGE
USE SEARCH
NO. OF SLICES= 40
NO. OF ADD. RADII= 5
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF CENTERS TO BE ANLYZED= 1

ONLY F. S. AT EACH CENTER WILL BE PRINTED
SLICES WILL BE SUBDIVIDED

SEARCH STARTED AT CENTER NO. 1
X COORDINATE= 180   Y COORDINATE= 410
X INCREMENT = 5   Y INCREMENT = 5

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE
MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X	CENTER Y	NO. OF CIRCLE			LOWEST	WARNING
COORDINATE	COORDINATE	TOTAL	CRITIC.	RADIUS	F.S.	
180	410	25	1	150.000	2.119	0
185	410	25	1	150.000	2.100	0
190	410	25	1	150.000	2.088	0
195	410	25	1	147.340	2.104	1
190	415	25	1	154.237	2.105	1
190	405	25	1	145.000	2.078	0
190	400	25	1	140.000	2.068	0
190	395	25	1	135.000	2.059	0
190	390	25	1	130.000	2.052	0
190	385	25	1	125.000	2.046	0
190	380	25	1	120.000	2.042	0
190	375	25	1	115.000	2.041	0
190	370	25	1	110.000	2.042	0
195	375	25	1	115.000	2.035	0
200	375	25	1	115.000	2.041	0
195	380	25	1	120.000	2.037	0
195	370	25	1	110.000	2.036	0
196.25	375	25	1	115.000	2.036	0
193.75	375	25	1	115.000	2.036	0
195	376.25	25	1	116.250	2.036	0
195	373.75	25	1	113.750	2.035	0

195	372.5	25	1	112.500	2.035	0
195	371.25	25	1	111.250	2.035	0
196.25	372.5	25	1	112.500	2.036	0
193.75	372.5	25	1	112.500	2.035	0

AT POINT (195 372.5) RADIUS 112.500

THE MINIMUM FACTOR OF SAFETY IS 2.035

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL SLIP SURFACE

SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	SLICE SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	2	3.170	1.798	0.000	-.743	0.651E+03	0.651E+03	0.762E+05	-.545E+05
2	2	0.310	3.679	0.000	-.728	0.130E+03	0.130E+03	0.977E+04	-.107E+05
3	1	1.273	4.475	0.000	-.720	0.647E+03	0.647E+03	0.258E+06	-.525E+05
4	1	4.753	7.358	0.000	-.694	0.392E+04	0.392E+04	0.928E+06	-.306E+06
5	1	4.753	11.475	0.000	-.651	0.606E+04	0.606E+04	0.881E+06	-.444E+06
6	1	4.753	15.129	0.000	-.609	0.796E+04	0.796E+04	0.843E+06	-.546E+06
7	1	4.753	18.381	0.000	-.567	0.966E+04	0.966E+04	0.811E+06	-.616E+06
8	1	2.404	20.589	0.000	-.535	0.546E+04	0.546E+04	0.400E+06	-.329E+06
9	1	2.349	21.707	0.000	-.514	0.562E+04	0.562E+04	0.385E+06	-.325E+06
10	1	1.651	22.378	0.000	-.496	0.407E+04	0.407E+04	0.267E+06	-.227E+06
11	1	3.102	24.014	0.000	-.475	0.819E+04	0.819E+04	0.496E+06	-.438E+06
12	1	4.753	27.359	0.000	-.440	0.143E+05	0.143E+05	0.744E+06	-.709E+06
13	1	4.753	31.154	0.000	-.398	0.163E+05	0.163E+05	0.729E+06	-.730E+06
14	1	4.753	34.689	0.000	-.356	0.182E+05	0.182E+05	0.715E+06	-.726E+06
15	1	4.753	37.977	0.000	-.313	0.199E+05	0.199E+05	0.704E+06	-.701E+06
16	1	4.753	41.031	0.000	-.271	0.215E+05	0.215E+05	0.694E+06	-.655E+06
17	1	4.753	43.860	0.000	-.229	0.230E+05	0.230E+05	0.687E+06	-.591E+06
18	1	4.753	46.470	0.000	-.187	0.243E+05	0.243E+05	0.680E+06	-.511E+06
19	1	4.753	48.869	0.000	-.144	0.256E+05	0.256E+05	0.676E+06	-.416E+06
20	1	4.753	51.060	0.000	-.102	0.267E+05	0.267E+05	0.672E+06	-.307E+06
21	1	4.753	53.048	0.000	-.060	0.278E+05	0.278E+05	0.670E+06	-.187E+06
22	1	4.753	54.833	0.000	-.018	0.287E+05	0.287E+05	0.669E+06	-.571E+05
23	1	4.753	56.417	0.000	0.025	0.296E+05	0.296E+05	0.669E+06	0.817E+05
24	1	4.753	57.800	0.000	0.067	0.303E+05	0.303E+05	0.670E+06	0.228E+06
25	1	4.753	58.981	0.000	0.109	0.309E+05	0.309E+05	0.672E+06	0.379E+06
26	1	4.753	59.957	0.000	0.151	0.314E+05	0.314E+05	0.676E+06	0.535E+06
27	1	4.753	60.726	0.000	0.194	0.318E+05	0.318E+05	0.681E+06	0.693E+06
28	1	4.753	61.281	0.000	0.236	0.321E+05	0.321E+05	0.688E+06	0.852E+06
29	1	4.753	61.618	0.000	0.278	0.323E+05	0.323E+05	0.696E+06	0.101E+07
30	1	4.753	61.727	0.000	0.320	0.324E+05	0.324E+05	0.706E+06	0.117E+07
31	1	4.588	61.605	0.000	0.362	0.312E+05	0.312E+05	0.692E+06	0.127E+07
32	1	0.166	61.429	0.000	0.383	0.112E+04	0.112E+04	0.252E+05	0.484E+05
33	1	4.753	60.528	0.000	0.405	0.317E+05	0.317E+05	0.731E+06	0.145E+07
34	1	4.753	58.587	0.000	0.447	0.307E+05	0.307E+05	0.747E+06	0.154E+07
35	1	4.753	56.365	0.000	0.489	0.296E+05	0.296E+05	0.766E+06	0.163E+07
36	1	4.753	53.839	0.000	0.532	0.282E+05	0.282E+05	0.789E+06	0.169E+07
37	1	4.753	50.982	0.000	0.574	0.267E+05	0.267E+05	0.816E+06	0.173E+07
38	1	4.753	47.759	0.000	0.616	0.251E+05	0.251E+05	0.849E+06	0.174E+07
39	1	3.315	44.703	0.000	0.652	0.164E+05	0.164E+05	0.615E+06	0.120E+07
40	1	1.438	42.704	0.000	0.673	0.678E+04	0.678E+04	0.273E+06	0.514E+06
41	1	4.753	39.778	0.000	0.701	0.209E+05	0.209E+05	0.937E+06	0.165E+07
42	1	4.753	34.817	0.000	0.743	0.183E+05	0.183E+05	0.998E+06	0.153E+07
43	1	4.753	29.183	0.000	0.785	0.153E+05	0.153E+05	0.108E+07	0.135E+07
44	1	4.753	22.694	0.000	0.827	0.119E+05	0.119E+05	0.119E+07	0.111E+07
45	1	4.753	15.053	0.000	0.870	0.794E+04	0.794E+04	0.135E+07	0.777E+06
46	1	3.519	7.036	0.000	0.906	0.277E+04	0.277E+04	0.117E+07	0.283E+06

```
47   2  1.234   1.564      0.000  0.927 0.221E+03 0.221E+03 0.415E+05 0.230E+05
                                                  SUM       0.315E+08 0.155E+08

AT CENTER (  195.000,    372.500) WITH RADIUS    112.500 AND SEISMIC COEFF.  0.00
FACTOR OF SAFETY BY NORMAL METHOD IS     2.030
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS    2.035
```

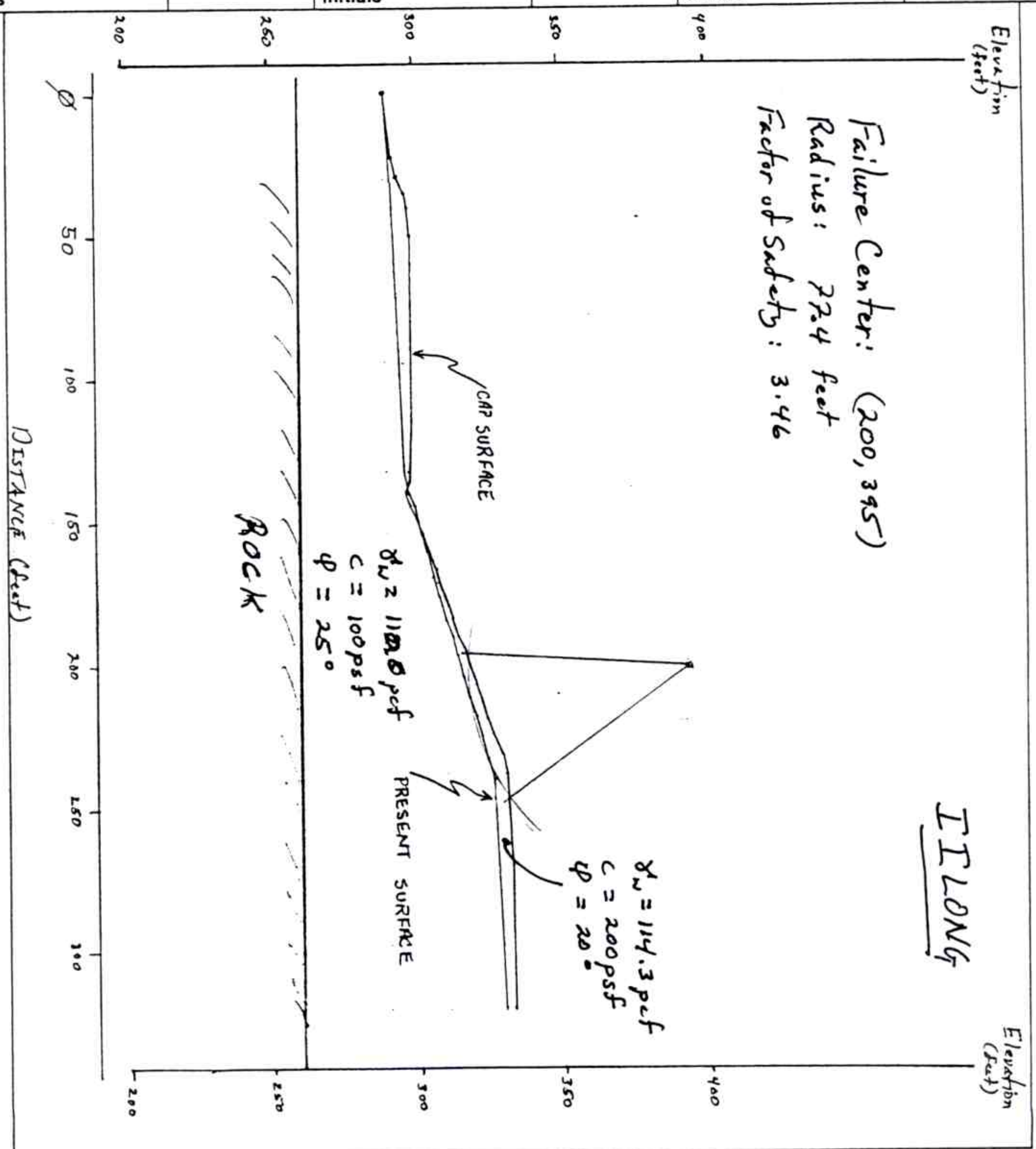
COMPUTATION SHEET

OHM Corporation

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject	Page ___ of ___
16139	Proteco	Puerto Rico	Slope Stability	

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date



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INPUT FILE NAME -c:\16139\iilong2.dta

TITLE -iilong2.dta

NO. OF STATIC AND SEISMIC CASES- 1

NO. OF NONCIRCULAR SLIP SURFACES= 0


CASE NO. 1    SEISMIC COEFFICIENT= 0

NO. OF BOUNDARY LINES= 3

NO. OF POINTS ON BOUNDARY LINE 1 = 2
 1  X COORD.= 0                Y COORD.= 260
 2  X COORD.= 320              Y COORD.= 260

NO. OF POINTS ON BOUNDARY LINE 2 = 5
 1  X COORD.= 0                Y COORD.= 290
 2  X COORD.= 132              Y COORD.= 296
 3  X COORD.= 140              Y COORD.= 297
 4  X COORD.= 240              Y COORD.= 326
 5  X COORD.= 320              Y COORD.= 330

NO. OF POINTS ON BOUNDARY LINE 3 = 10
 1  X COORD.= 0                Y COORD.= 290
 2  X COORD.= 23               Y COORD.= 292
 3  X COORD.= 40               Y COORD.= 298
 4  X COORD.= 50               Y COORD.= 299
 5  X COORD.= 113              Y COORD.= 299
 6  X COORD.= 136              Y COORD.= 298
 7  X COORD.= 140              Y COORD.= 297
 8  X COORD.= 238              Y COORD.= 330
 9  X COORD.= 270              Y COORD.= 332
 10 X COORD.= 320              Y COORD.= 332

LINE NO. AND SLOPE OF EACH SEGMENT ARE:
 1        +0.000
 2        +0.045     +0.125     +0.290     +0.050
 3        +0.087     +0.353     +0.100     +0.000     -0.043     -0.250
          +0.337     +0.063     +0.000

MIN. DEPTH OF TALLEST SLICE= 0
NO. OF RADIUS CONTROL ZONES= 1

RADIUS DECREMENT FOR ZONE 1 = 0
NO. OF CIRCLES FOR ZONE 1 = 10
ID NO. FOR FIRST CIRCLE FOR ZONE 1 = 1
NO. OF BOTTOM LINES FOR ZONE 1 = 1

FOR ZONE 1     LINE SEQUENCE 1
LINE NO.= 2    BEG. NO.= 1   END NO.= 5
UNIT WEIGHT OF WATER= 62.4
```

SOIL NO.	COHESION	FRIC. ANGLE	UNIT WEIGHT
1	100	25	110
2	200	20	114.3

NO SEEPAGE
USE SEARCH
NO. OF SLICES= 20
NO. OF ADD. RADII= 5
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF CENTERS TO BE ANLYZED= 1

ONLY F. S. AT EACH CENTER WILL BE PRINTED
SLICES WILL NOT BE SUBDIVIDED

SEARCH STARTED AT CENTER NO. 1
X COORDINATE= 250 Y COORDINATE= 400
X INCREMENT = 5 Y INCREMENT = 5

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
250	400	10	1	73.408	14.236	0
255	400	10	1	73.159	18.293	0
245	400	10	1	73.658	11.382	0
240	400	10	1	73.908	9.528	0
235	400	10	1	74.169	8.145	0
230	400	10	1	74.673	6.908	0
225	400	10	1	75.505	5.816	0
220	400	10	1	76.655	4.929	0
215	400	10	1	78.035	4.262	0
210	400	10	1	79.427	3.839	0
205	400	10	1	80.820	3.592	0
200	400	10	1	82.213	3.473	0
195	400	10	1	83.605	3.481	0
200	405	10	1	87.015	3.503	0
200	395	10	1	77.411	3.461	0
200	390	10	1	72.608	3.464	0
205	395	10	1	76.018	3.533	0
195	395	10	1	78.803	3.527	0
201.25	395	10	1	77.062	3.463	0
198.75	395	10	1	77.759	3.463	0
200	396.25	10	1	78.611	3.461	0
200	393.75	10	1	76.210	3.461	0

AT POINT (200 395) RADIUS 77.411

THE MINIMUM FACTOR OF SAFETY IS 3.461

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL SLIP SURFACE

SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	SLICE SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	2	2.065	0.325	0.000	0.028	0.767E+02	0.767E+02	0.341E+05	0.169E+03
2	2	2.065	0.934	0.000	0.055	0.220E+03	0.220E+03	0.382E+05	0.941E+03
3	2	2.065	1.488	0.000	0.082	0.351E+03	0.351E+03	0.419E+05	0.222E+04
4	2	2.065	1.986	0.000	0.108	0.469E+03	0.469E+03	0.453E+05	0.394E+04
5	2	2.065	2.428	0.000	0.135	0.573E+03	0.573E+03	0.483E+05	0.599E+04
6	2	2.065	2.813	0.000	0.162	0.664E+03	0.664E+03	0.509E+05	0.832E+04
7	2	2.065	3.141	0.000	0.189	0.741E+03	0.741E+03	0.531E+05	0.108E+05
8	2	2.065	3.410	0.000	0.215	0.805E+03	0.805E+03	0.549E+05	0.134E+05
9	2	2.065	3.621	0.000	0.242	0.855E+03	0.855E+03	0.563E+05	0.160E+05
10	2	2.065	3.771	0.000	0.269	0.890E+03	0.890E+03	0.573E+05	0.185E+05
11	2	2.065	3.860	0.000	0.295	0.911E+03	0.911E+03	0.580E+05	0.208E+05
12	2	2.065	3.885	0.000	0.322	0.917E+03	0.917E+03	0.582E+05	0.228E+05
13	2	2.065	3.846	0.000	0.349	0.908E+03	0.908E+03	0.581E+05	0.245E+05
14	2	2.065	3.739	0.000	0.375	0.883E+03	0.883E+03	0.575E+05	0.256E+05
15	2	2.065	3.564	0.000	0.402	0.841E+03	0.841E+03	0.566E+05	0.262E+05
16	2	2.065	3.317	0.000	0.429	0.783E+03	0.783E+03	0.553E+05	0.260E+05
17	2	2.065	2.994	0.000	0.455	0.707E+03	0.707E+03	0.536E+05	0.249E+05
18	2	2.065	2.594	0.000	0.482	0.612E+03	0.612E+03	0.516E+05	0.228E+05
19	2	2.065	1.736	0.000	0.509	0.410E+03	0.410E+03	0.471E+05	0.161E+05
20	2	2.065	0.601	0.000	0.535	0.142E+03	0.142E+03	0.412E+05	0.588E+04
							SUM	0.102E+07	0.296E+06

AT CENTER (200.000, 395.000) WITH RADIUS 77.411 AND SEISMIC COEFF. 0.00
FACTOR OF SAFETY BY NORMAL METHOD IS 3.438
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS 3.461

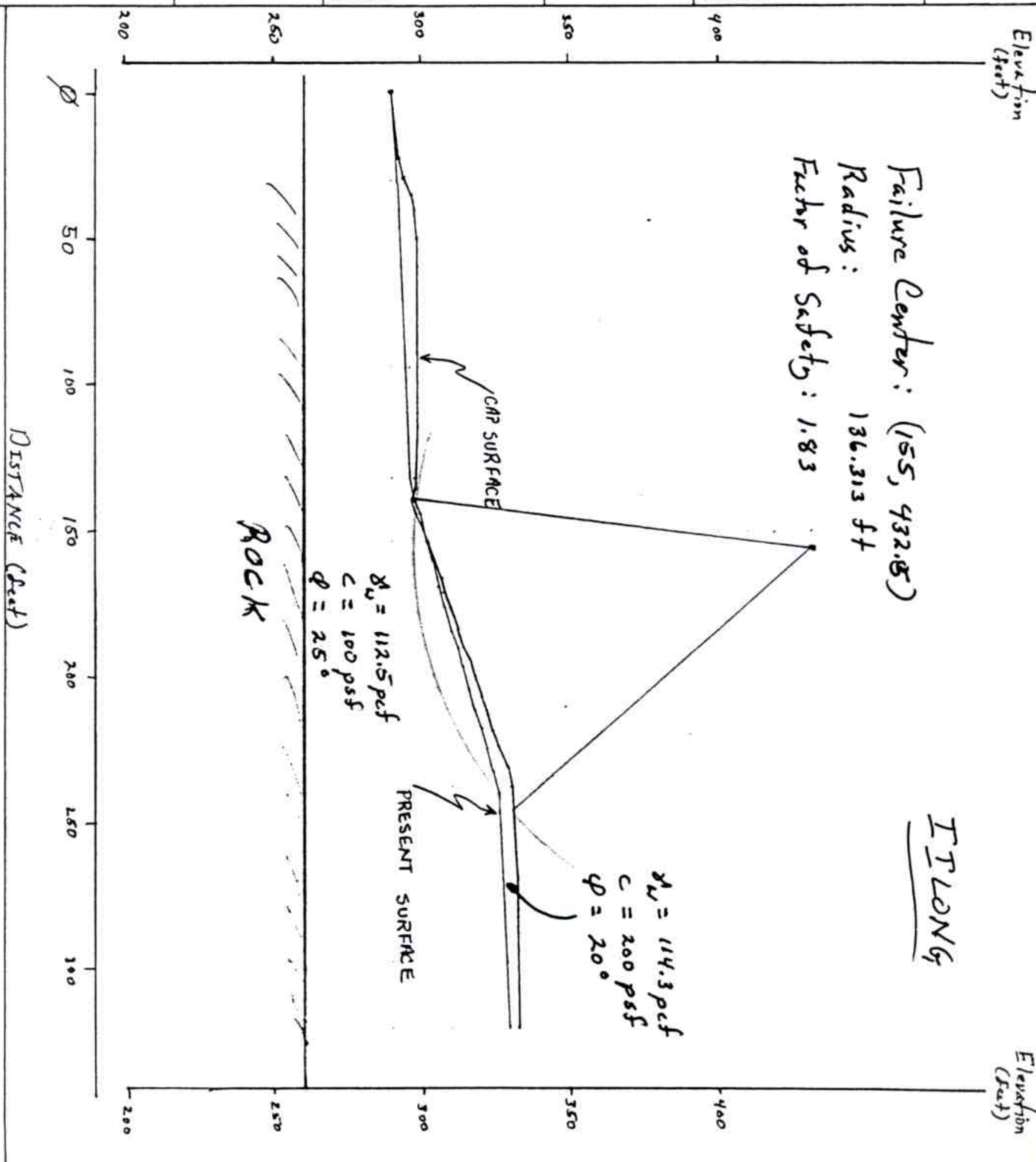
OHM Corporation

COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
16139	Proteco	Puerto Rico	Slope Stability

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date



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UNIVERSITY OF KENTUCKY, LEXINGTON, KY 40506

INPUT FILE NAME -c:\16139\iilong.dta

TITLE -iilong.dta

NO. OF STATIC AND SEISMIC CASES- 1

NO. OF NONCIRCULAR SLIP SURFACES= 0


CASE NO. 1    SEISMIC COEFFICIENT= 0

NO. OF BOUNDARY LINES= 3

NO. OF POINTS ON BOUNDARY LINE 1 = 2
 1  X COORD.= 0              Y COORD.= 270
 2  X COORD.= 320            Y COORD.= 270

NO. OF POINTS ON BOUNDARY LINE 2 = 5
 1  X COORD.= 0              Y COORD.= 290
 2  X COORD.= 132            Y COORD.= 296
 3  X COORD.= 140            Y COORD.= 297
 4  X COORD.= 240            Y COORD.= 326
 5  X COORD.= 320            Y COORD.= 330

NO. OF POINTS ON BOUNDARY LINE 3 = 10
 1  X COORD.= 0              Y COORD.= 290
 2  X COORD.= 23             Y COORD.= 292
 3  X COORD.= 40             Y COORD.= 298
 4  X COORD.= 50             Y COORD.= 299
 5  X COORD.= 113            Y COORD.= 299
 6  X COORD.= 136            Y COORD.= 298
 7  X COORD.= 140            Y COORD.= 297
 8  X COORD.= 238            Y COORD.= 330
 9  X COORD.= 270            Y COORD.= 332
 10 X COORD.= 320            Y COORD.= 332

LINE NO. AND SLOPE OF EACH SEGMENT ARE:
 1        +0.000
 2        +0.045     +0.125     +0.290     +0.050
 3        +0.087     +0.353     +0.100     +0.000     -0.043     -0.250
          +0.337     +0.063     +0.000

MIN. DEPTH OF TALLEST SLICE= 0
NO. OF RADIUS CONTROL ZONES= 2

RADIUS DECREMENT FOR ZONE 1 = 0
NO. OF CIRCLES FOR ZONE 1 = 10
ID NO. FOR FIRST CIRCLE FOR ZONE 1 = 1
NO. OF BOTTOM LINES FOR ZONE 1 = 1

FOR ZONE 1    LINE SEQUENCE 1
LINE NO.= 1   BEG. NO.= 1   END NO.= 2

RADIUS DECREMENT FOR ZONE 2 = 0
```

NO. OF CIRCLES FOR ZONE 2 = 5
ID NO. FOR FIRST CIRCLE FOR ZONE 2 = 1
NO. OF BOTTOM LINES FOR ZONE 2 = 1

FOR ZONE 2 LINE SEQUENCE 1
LINE NO.= 2 BEG. NO.= 1 END NO.= 5
UNIT WEIGHT OF WATER= 62.4

SOIL NO.	COHESION	FRIC. ANGLE	UNIT WEIGHT
1	100	25	112.5
2	200	20	114.3

NO SEEPAGE
USE SEARCH
NO. OF SLICES= 20
NO. OF ADD. RADII= 5
ANALYSIS BY SIMPLIFIED BISHOP METHOD (MTHD=2)
NUMBER OF FORCES (NFO)= 0
SOFT SOIL NUMBER (SSN)= 0

NO. OF CENTERS TO BE ANLYZED= 1

ONLY F. S. AT EACH CENTER WILL BE PRINTED
SLICES WILL BE SUBDIVIDED

SEARCH STARTED AT CENTER NO. 1
X COORDINATE= 180 Y COORDINATE= 300
X INCREMENT = 5 Y INCREMENT = 5

IN THE FOLLOWING TABLE WARNING INDICATES HOW MANY TIMES THE
MAXIMUM RADIUS IS LIMITED BY THE END POINTS OF GROUND LINES

CENTER X COORDINATE	CENTER Y COORDINATE	NO. OF CIRCLE TOTAL	CRITIC.	RADIUS	LOWEST F.S.	WARNING
180	300	1	1	30.000	1000.000	0
185	300	1	1	30.000	1000.000	0
175	300	2	1	30.000	6.197	0
170	300	4	1	30.000	5.834	0
165	300	16	13	26.544	5.701	0
160	300	17	13	21.352	5.642	0
155	300	19	14	16.127	5.543	0
150	300	21	18	11.061	5.359	0
145	300	23	21	5.765	5.026	0
140	300	10	1	30.000	12.724	0
145	305	24	19	9.640	3.730	0
145	310	24	10	13.984	3.413	0
145	315	24	20	18.321	3.368	0
145	320	25	21	23.139	3.248	0
145	325	25	23	29.925	3.308	0
150	320	25	21	24.932	2.607	0
155	320	25	21	27.004	2.565	0
160	320	25	22	25.448	2.632	0
155	325	25	22	31.324	2.432	0
155	330	25	17	36.179	2.339	0
155	335	25	18	40.489	2.281	0

155	340	25	18	45.340	2.228	0
155	345	25	18	50.192	2.186	0
155	350	25	18	55.044	2.151	0
155	355	25	18	59.895	2.122	0
155	360	25	18	64.747	2.097	0
155	365	25	18	69.598	2.075	0
155	370	25	18	74.450	2.055	0
155	375	25	18	79.302	2.038	0
155	380	25	18	84.153	2.022	0
155	385	25	18	89.005	2.008	0
155	390	25	18	93.857	1.994	0
155	395	25	18	98.708	1.982	0
155	400	25	18	103.560	1.971	0
155	405	25	17	109.002	1.944	0
155	410	25	22	113.857	1.934	0
155	415	25	22	118.712	1.925	0
155	420	25	22	123.567	1.912	0
155	425	25	22	128.422	1.904	0
155	430	25	22	133.277	1.900	0
155	435	25	15	138.742	1.892	0
155	440	25	15	143.600	1.897	0
160	435	25	20	139.012	1.912	0
150	435	25	19	137.978	1.935	0
156.25	435	25	21	138.493	1.897	0
153.75	435	25	17	138.387	1.897	0
155	436.25	25	15	139.957	1.893	0
155	433.75	25	15	137.528	1.891	0
155	432.5	25	15	136.313	1.890	0
155	431.25	25	17	134.491	1.899	0
156.25	432.5	25	21	136.064	1.895	0
153.75	432.5	25	17	135.960	1.898	0

AT POINT (155 432.5) RADIUS 136.313

THE MINIMUM FACTOR OF SAFETY IS 1.890

SUMMARY OF SLICE INFORMATION FOR MOST CRITICAL SLIP SURFACE

SL. NO.	SOIL NO.	SLICE WIDTH	SLICE HEIGHT	WATER HEIGHT	SLICE SINE	TOTAL WEIGHT	EFFEC. WEIGHT	RESIS. MOMENT	DRIVING MOMENT
1	1	5.268	1.152	0.000	-.090	0.684E+03	0.684E+03	0.115E+06	-.843E+04
2	1	5.268	3.302	0.000	-.052	0.196E+04	0.196E+04	0.196E+06	-.138E+05
3	1	5.268	5.247	0.000	-.013	0.312E+04	0.312E+04	0.270E+06	-.560E+04
4	1	5.268	6.988	0.000	0.025	0.415E+04	0.415E+04	0.335E+06	0.144E+05
5	1	5.268	8.526	0.000	0.064	0.506E+04	0.506E+04	0.393E+06	0.442E+05
6	1	5.268	9.859	0.000	0.103	0.586E+04	0.586E+04	0.442E+06	0.820E+05
7	1	5.268	10.985	0.000	0.141	0.652E+04	0.652E+04	0.483E+06	0.126E+06
8	1	5.268	11.901	0.000	0.180	0.707E+04	0.707E+04	0.515E+06	0.173E+06
9	1	5.268	12.603	0.000	0.219	0.749E+04	0.749E+04	0.538E+06	0.223E+06
10	1	5.268	13.085	0.000	0.257	0.778E+04	0.778E+04	0.552E+06	0.273E+06
11	1	5.268	13.342	0.000	0.296	0.793E+04	0.793E+04	0.557E+06	0.320E+06
12	1	5.268	13.366	0.000	0.335	0.795E+04	0.795E+04	0.552E+06	0.362E+06
13	1	5.268	13.146	0.000	0.373	0.782E+04	0.782E+04	0.539E+06	0.398E+06
14	1	5.268	12.671	0.000	0.412	0.754E+04	0.754E+04	0.516E+06	0.423E+06
15	1	5.268	11.926	0.000	0.451	0.710E+04	0.710E+04	0.483E+06	0.436E+06
16	1	5.268	10.895	0.000	0.489	0.649E+04	0.649E+04	0.442E+06	0.433E+06
17	1	5.268	9.557	0.000	0.528	0.570E+04	0.570E+04	0.392E+06	0.410E+06
18	1	5.268	7.887	0.000	0.566	0.471E+04	0.471E+04	0.334E+06	0.364E+06
19	1	3.150	6.292	0.000	0.597	0.226E+04	0.226E+04	0.169E+06	0.184E+06

20	1	2.117	4.876	0.000	0.617	0.118E+04	0.118E+04	0.956E+05	0.990E+05
21	2	5.268	2.109	0.000	0.644	0.127E+04	0.127E+04	0.236E+06	0.111E+06
							SUM	0.816E+07	0.445E+07

AT CENTER (155.000, 432.500) WITH RADIUS 136.313 AND SEISMIC COEFF. 0.00
FACTOR OF SAFETY BY NORMAL METHOD IS 1.833
FACTOR OF SAFETY BY SIMPLIFIED BISHOP METHOD IS 1.890

Page 1 of					
Proj. No. 16139	Client PROTECO	Location Peñuelas, Puerto Rico		Subject RCRA Cap Stability	
Preparer's Initials Jac	Date 9-23-94	Reviewer's Initials BB	Date 9/26/94	Approver's Initials	Date

Determine: The safety factor for the stability of the RCRA Cap, i.e. sliding stability of the geosynthetics.

Given: GeoSyntec's Test Results (see GeoSyntec's Report for details)

Interface Direct Shear Test Results

1. Geonet / 20-mil PVC Geomembrane
   Peak value $\delta_P = 12^\circ$
   Residual value $\delta_R = 12^\circ$

2. Clay liner / 20-mil PVC Geomembrane
   Peak value $\delta_P = 14^\circ$
   Residual value $\delta_R = 5^\circ$

Solution:

Clay liner / 20-PVC Geomembrane controls the design

$SF = \dfrac{\tan \delta}{\tan i}$ (Soil Mechanics, Lambe & Whitman, John Wiley & Sons, 1969, pg 193)

where: δ = Interface Friction for Clay liner / Geomembrane
i = Cap slope = 4.57° (8% is the maximum cap grade)

$SF = \tan 14^\circ / \tan 4.57^\circ = 3.1$ for peak value

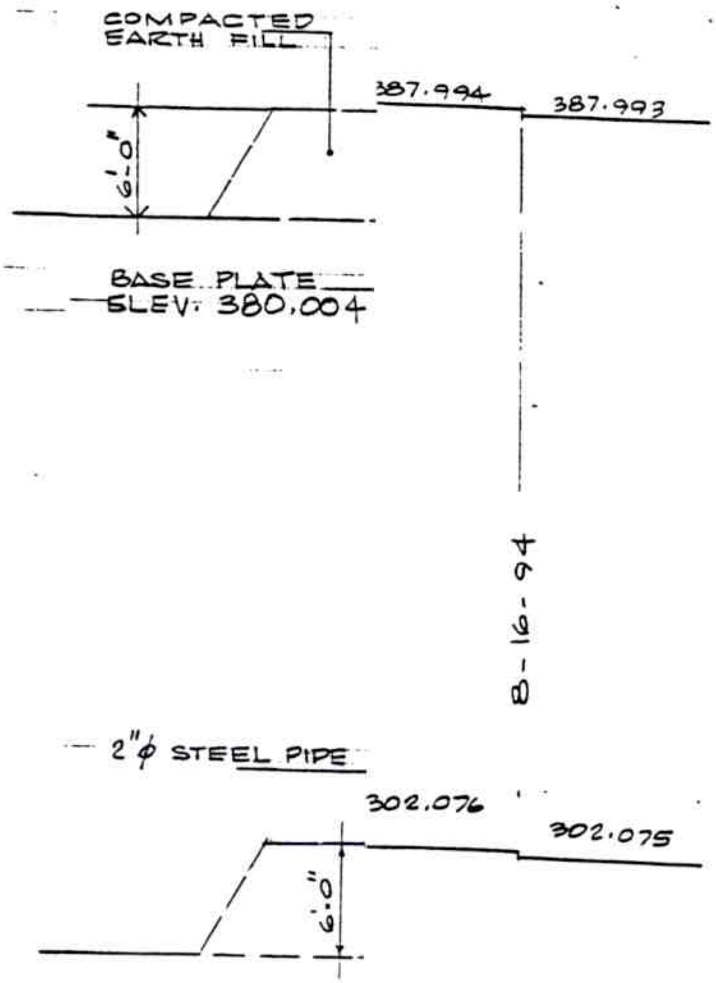
$SF = \tan 5^\circ / \tan 4.57^\circ = 1.1$ for residual value

∴ The SF to sliding is 3.1. Even if movement occurs, the SF is 1.1, which is adequate for such a condition.

Note: Values of interface shear strength will be verified for the actual geosynthetics used for the project.

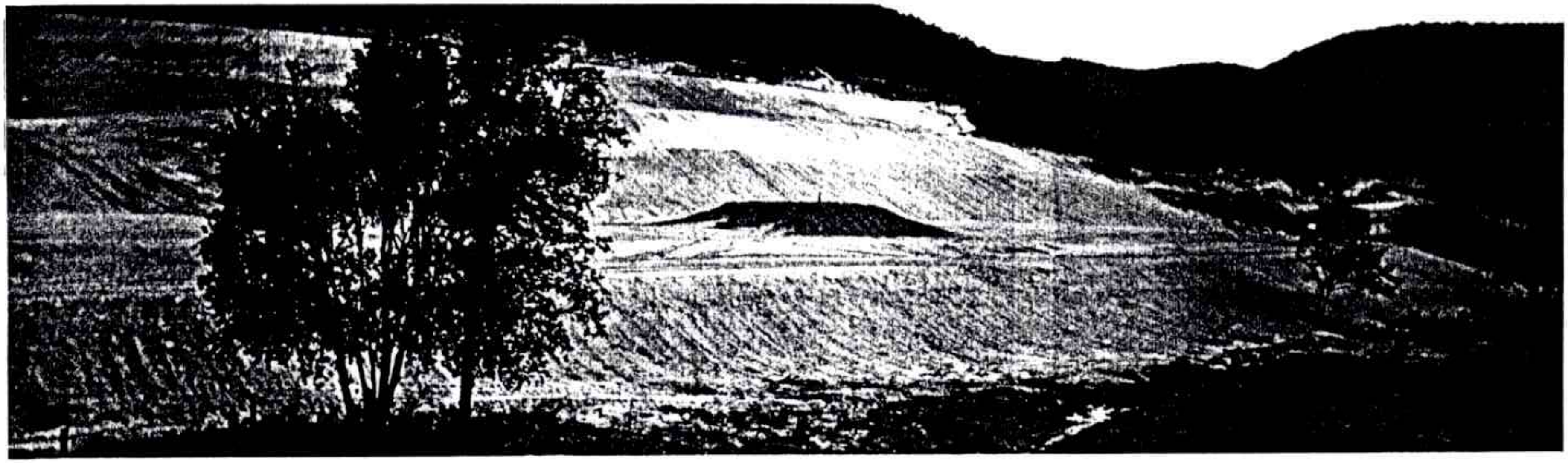
APPENDIX I

SETTLEMENT DATA FOR WASTE UNITS 1 AND 16


COMPACTED EARTH FILL
387.994
387.993
6'-0"
BASE PLATE
ELEV. 380.004
B-16-94
2"ϕ STEEL PIPE
302.076
302.075
6'-0"

DIAGRAMS

Field Load Test Unit No. 1 - PROTECO Hazardous Waste Units
Penuelas, Puerto Rico - August 24, 1994, Project No. 16139



Field Load Test Unit No. 16 - PROTECO Hazardous Waste Units
Penuelas, Puerto Rico - August 24, 1994, Project No. 16139

APPENDIX J

GEOMEMBRANE STRENGTH AND STABILITY CALCULATIONS

OHM Remediation Services Corp. **COMPUTATION SHEET**

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
18146	PROTECO	PENUELAS, P.R.	GEOMEMBRANE E

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JKO	11/13/95	JB	11/14/95		

ESTIMATE ABILITY OF GEOMEMBRANE CAP TO SURVIVE SETTLEMENT RESULTING FROM LONG TERM WASTE COMPRESSION.

GIVEN:

LET MAXIMUM WASTE DEPTH = 20 FT
SETTLEMENT (S) = 10%
SETTLEMENT RATIO (SR) = D/W
WIDTH OF CELL (W) MAX = 100 FT

STRAIN FAILURE FOR HDPE (E) (CADWALLER, 1990) = 107%

SOLUTION:

ESTIMATE THE GEOMETRY OF SETTLEMENT DEPRESSION

SETTLEMENT DEPTH = D * S = 20ft (0.1%)
SETTLEMENT DEPTH = 2ft

SETTLEMENT RATIO = $\frac{2ft}{100ft}$ = 0.02 ✓

OBTAIN UNIFORM STRAIN FROM CIRCULAR TROUGH MODEL. (SEE ATTACHED FIGURE)

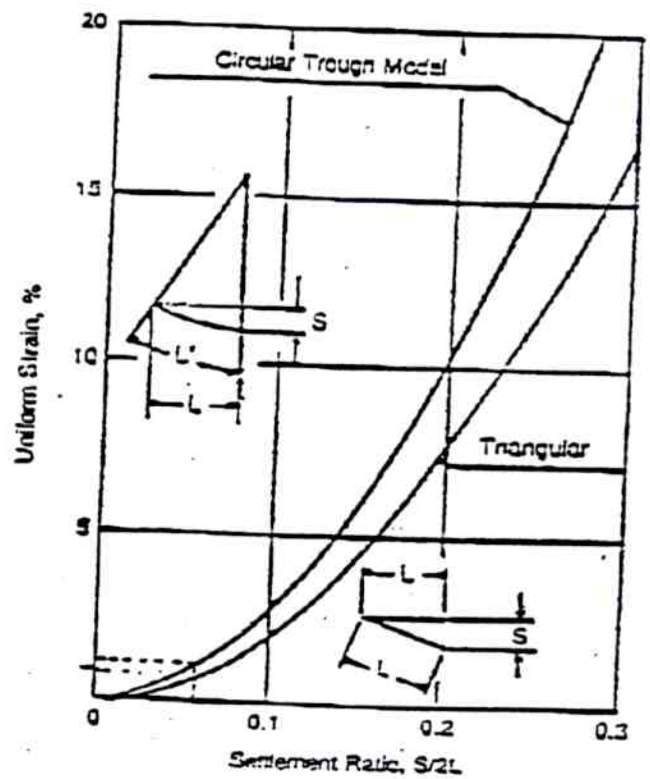
ϵ = 1% ✓ from table

Design Ratio = $\frac{\epsilon_{rupture}}{\epsilon_{uniform}} = \frac{107\%}{1\%} = 107$ ✓

DR must be greater than 5 OK

No failure

2/2
GEOMEMBRANE E



Settlement trough models (Knipschield, 1985).

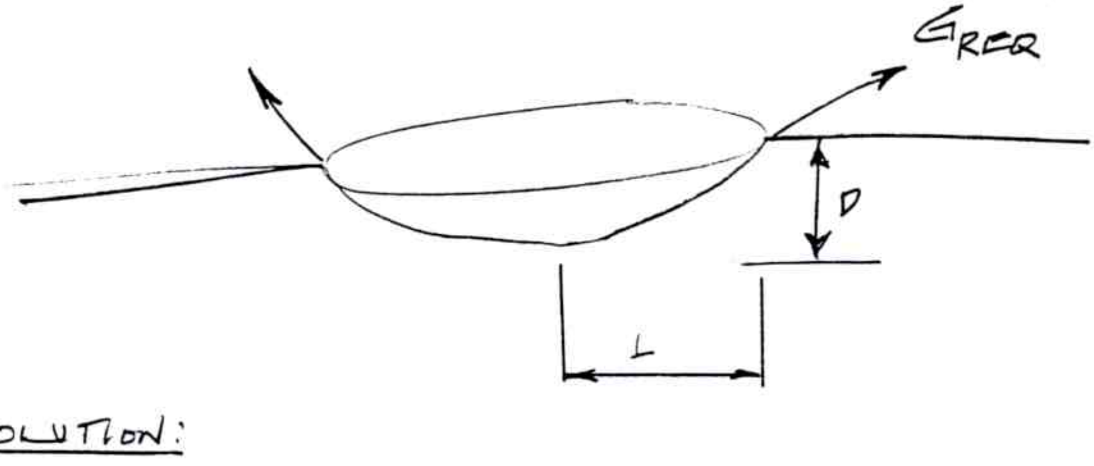
Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 18146	Client PROTECO	Location PENUELAS, P.R.	Subject δ OF GEOMEMBRANE
Preparer's Initials JKO	Date 11/13/95	Reviewer's Initials [initials] Date 11/14/95	Approver's Initials Date

VERIFY THE ABILITY OF THE GEOMEMBRANE CAP TO SURVIVE DIFFERENTIAL SETTLEMENT.

GIVEN:

γ = UNIT WEIGHT OF COVER SOIL = 120 pcf
H = HEIGHT OF COVER = 4 ft
t = GEOMEMBRANE THICKNESS = 40mil = 0.04in
D = (See sketch) = 1 ft (assumed EPA/625/4-91/025)
L = (See sketch) = 3 ft (" ")
σ = STRENGTH $\Rightarrow$ σ_y = 2200 psi for HDPE
FS = FACTOR OF SAFETY > 1



SOLUTION:

$$\sigma_{REQ} = \frac{2DL^2\gamma H}{3t(D^2+L^2)}$$

$$= \frac{2(1ft)(3ft)^2\,120\,lb/ft^3\,(4ft)}{3\left(0.04in\cdot\frac{ft}{12in}\right)(3ft^2+1ft^2)}$$

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COMPUTATION SHEET

Form No. 0048
Midwest Tech. Servs.
Rev. 08/89

Proj. No.	Client	Location	Subject
18146	PROTECO	PENUELAS, P.R.	σ OF GEOMEMBRANE

Preparer's Initials	Date	Reviewer's Initials	Date	Approver's Initials	Date
JKD	11/13/95	JBR	11/14/95		

$\sigma_{REQ} = 86,400 \text{ psf} = 600 \text{ psi}$

$$FS = \frac{\sigma_{ALLOWABLE}}{\sigma_{REQ}} = \frac{2200 \text{ psi}}{600 \text{ psi}}$$

$FS = 3.67 > 1$ OK

MEMBRANE WILL SURVIVE CONDITIONS MODELED

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COMPUTATION SHEET

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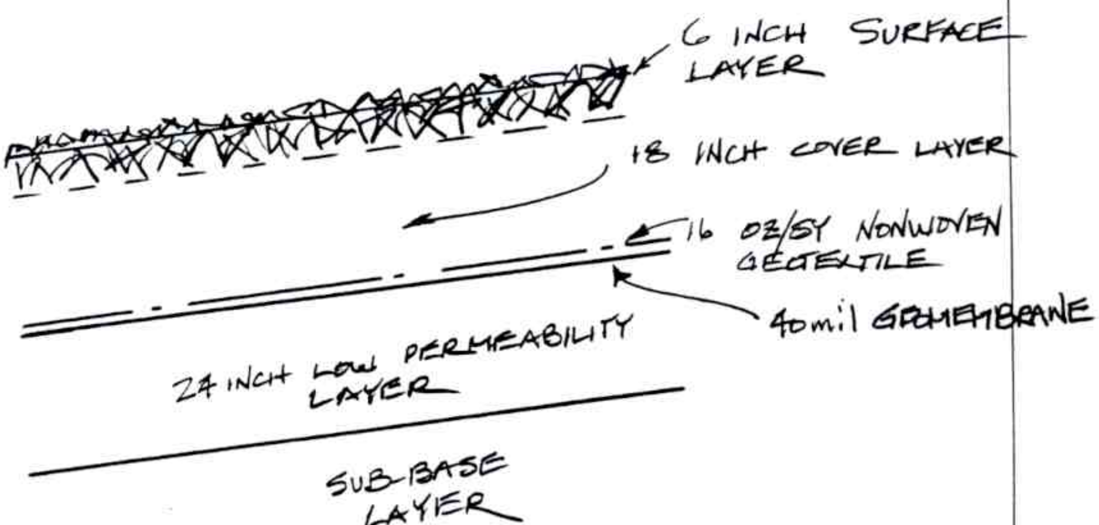
Rev. 08/89

Proj. No.	Client	Location	Subject	
18146	PROTECO	PENUELAS, P.R.	COVER STABILITY	
Preparer's Initials: JKO	Date: 11/13/95	Reviewer's Initials: JER	Date: 11/14/95	Approver's Initials: Date:

ESTIMATE ABILITY OF SOIL COVER LAYER TO REMAIN ON THE GEOSYNTHETICS

GIVEN:

CONFIGURATION OF CAP IS SHOWN BELOW.



MATERIALS	ϕ, INTERFACE FRICTION
SOIL / GEOTEXTILE	25°
GEOTEXTILE / GEOMEMBRANE (SMOOTH)	9°
GEOMEMBRANE (SMOOTH) / SOIL	14°

NOTE: ADHESION FOR THE CLAY SOILS IS NEGLECTED, THEREFORE THE FACTOR OF SAFETY WILL BE UNDER REPORTED

FROM THE INTERFACE LISTED ABOVE THE CRITICAL INTERFACE IS BETWEEN THE GEOTEXTILE AND GEOMEMBRANE

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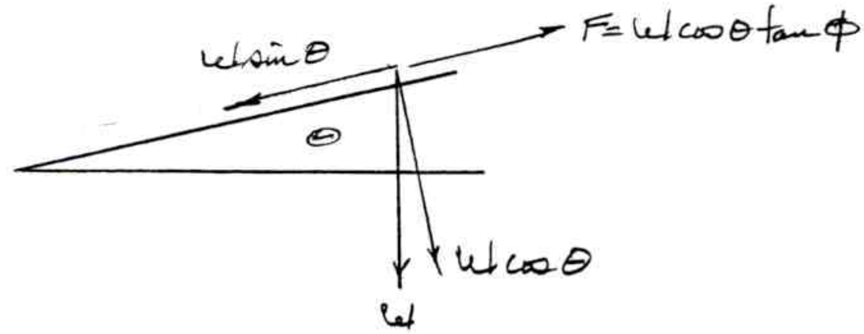
Midwest Tech. Servs.
Rev. 08/89

Proj. No. 18146	Client PROTECO	Location PENUELAS, P.R.	Subject COVER STABILITY
Preparer's Initials JKO	Date 11/13/95	Reviewer's Initials JEB	Date 11/14/95
Approver's Initials	Date		



$$FS = \frac{\cancel{W} \cos\theta \cdot \tan\phi}{\cancel{W} \sin\theta} = \frac{\tan\phi}{\tan\theta}$$

$\theta_{max} = 8°$ 30% of geomembrane

$\theta_{average} = 5°$ 70% of geomembrane

$$FS = \frac{\tan 9°}{\tan 8°} = 1.1 > 1 \checkmark \quad OK$$

$$FS = \frac{\tan 9°}{\tan 5°} = 1.8 > 1 \checkmark \quad OK$$

THEREFORE EARTH AND GEOSYNTHETICS WILL NOT SLIDE OFF THE COVER

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Form No. 0048
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Rev. 08/89

Proj. No. 18146	Client PROTECO	Location PENUELAS, P.R.	Subject GAS VOLUME
Preparer's Initials JKO	Date 11/13/95	Reviewer's Initials JED	Date 11/14/95	Approver's Initials	Date

Estimated Weight of Waste in Waste Units = W_{wt}

W_{wt} = V * Waste Density

$W_{wt} = (80{,}667\ yd^3) * 500\ lb/yd^3$

$W_{wt} = 40{,}333{,}500\ lb$ ✓

Total Volume of Gas Generated per Minute = G_T

$$G_T = \frac{(W_{wt} \times G_p)}{525{,}600\ \text{min}/\text{yr}}$$

$$G_T = \frac{40{,}333{,}500\ lb * 0.095\ ft^3/lb/yr}{525{,}600\ \text{min}/\text{yr}}$$

$G_T = 7.29\ ft^3/min$ ✓ Average gas generation for all waste units